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Assessment of Foreign Bridge Standards and Techniques

Anthony J. Lamanna, Mustafa Lok, Gerardo I. Velázquez, James C. Ray, and Terry R. Stanton

September 2004

Geotechnical and Structures Laboratory

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Assessment of Foreign Bridge Standards and Techniques

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ABSTRACT:

Turkish bridge design standards were studied, with attention focused on the live load. The design specifications were compared with American design specifications. The major difference was that the live load in Turkish standards is given in tonnes; whereas, in American Association of State Highway and Transportation Officials-Standard Specifications for Highway Bridges (AASHTO 1996) it is in tons. Therefore, HS20 in Turkish standards is 10 percent heavier than HS20-44. Turkish bridges are currently designed to either HS20 or HS30, the latter being 65 percent heavier than HS20-44. There were some minor differences in other requirements, due to conversion from United States customary units to metric units.

Three types of Turkish bridges were analyzed using a service load approach according to AASHTO (1996) using a Heavy Equipment Transporter as the live load. Service load approach was applied. Only the primary loads, dead load, live load, and impact were considered. The analysis did not include any modification for possible deterioration, damage, or aging of the bridges.

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Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI units as follows:

Multiply	By	To Obtain
cubic feet	0.028317	cubic meters
Fahrenheit degrees	5/9	Celsius degrees[1]
feet	0.304800	meters
feet per second	0.304800	meters per second
inches	25.4	millimeters
miles	1.609	kilometers
pounds (force) per square inch	0.006894757	megapascals
pounds (mass)	0.453592	kilograms

[1] To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use the following formula: C = (5/9) (F-32).
Note: The primary units presented in the report are those that were used during fabrication of experiment setup or those calibrated and recorded by instruments during experiment execution.

Preface

This study was conducted by personnel of the Department of Civil and Environmental Engineering, University of Tulane, New Orleans, LA, under contract No. DACA42-03-P-0010, and the U.S. Army Engineer Research and Development Center (ERDC), Geotechnical and Structures Laboratory (GSL), Vicksburg, MS. The study was part of the Department of the Army Project No. 4A162784AT40, Work Package 1259B, "Bridge Assessment and Repair," Work Unit BR002, "Rapid Bridge Repair and Retrofit," which is sponsored by Headquarters, U.S. Army Corps of Engineers.

The experimental work was accomplished under the general supervision of Dr. David W. Pittman, Acting Director, GSL, Mr. Andrew E. Jackson, Jr., Acting Deputy Director, GSL, Dr. Robert L. Hall, Chief, Geosciences and Structures Division (GSD), and Mr. James S. Shore, Chief, Structural Engineering Branch (StEB). Messrs. Gerardo I. Velázquez, James C. Ray, and Terry R. Stanton, StEB, were the project investigators for this effort. Dr. Anthony J. Lamanna was the Principal Investigator, and Mr. Mustafa Lok was the research assistant for Tulane University. Ms. Corine E. Pugh, Graphics Specialist/Web Author, Computer Science Corporation, also assisted the authors in the preparation of this report.

COL James R. Rowan, EN, was Commander and Executive Director of ERDC. Dr. James R. Houston was Director.

1 Introduction

Background

Forming a natural bridge between Asia and Europe, Turkey is the shortest connection between Europe and the Middle East. Therefore, the traffic through Turkey is important for both Turkey and other countries that use Turkish highways for transportation of equipment and goods.

Turkish construction practices have been used by Turkish contractors in Middle Eastern countries, in North Africa, and since the collapse of Soviet Union in the former Soviet Republics. These contractors also design and build bridges in these countries. Therefore, this study of Turkish bridge designs will increase knowledge of bridges in these countries as well.

The knowledge of the capacities of Turkish bridges will help businesses and governments more efficiently move equipment, personnel, and goods around the region by identifying maximum live loads along specific transportation routes.

Objectives

The objective of this research was to study the design of the bridges in Turkey. This objective was accomplished by studying Turkish Bridge specifications and design manuals particularly focusing on live loads.

Another objective was to analyze typical and specific bridges for a HET load. This objective was accomplished by studying three types of bridges (a reinforced concrete open-spandrel arch, a reinforced concrete T-Girder, and a composite steel I-Girder bridge) and by analyzing the superstructures of these bridges for HET loading.

Methodology and Scope

Bridge engineers in the General Directorate of Highways (GDH), the department responsible for the design, construction, operation, and maintenance of Turkish highways and bridges, were contacted. Design practices were discussed, and design specifications were obtained. These specifications were studied and compared with American Association of State Highway and

Transportation Officials-Standard Specifications for Highway Bridges (AASHTO 1996). The specifications for construction materials were also studied. In order to understand Turkish bridge design, blueprints of three types of Turkish bridges were obtained and studied.

These bridges were analyzed using Allowable Stress Design (ASD) approach for primary loads (dead load, live load, and impact) only. Neither the loads nor the section capacities were factored as the bridges were checked with service load approach for overloading. First-order analysis was used. Environmental factors, possible damages, cracks, deterioration, aging, and any vandalism that might have occurred were not considered in the analysis. Assumptions were made for any missing information for the analysis.

Bridge Selection

Three different types of Turkish bridges were selected. The first one is the Birecik Bridge, a reinforced concrete open-spandrel arch bridge on a state highway in southeast Turkey. The design drawings of the bridge could not be found. The external dimensions of the bridge were obtained through a Turkish survey team. The investigators analyzed this bridge with engineering assumptions. As there was no information about the reinforcement details, the amount of reinforcement assumed in the bridge was the minimum steel required by the American Concrete Institute (ACI). The minimum reinforcement requirements of the Turkish standards are the same as ACI. The analysis of this bridge is presented in Chapter 4.

The second bridge type was adopted from generic design plans prepared by the GDH. It is an I-Girder steel-concrete deck bridge. The substructure of the bridge was not analyzed as this project focused on the bridge superstructures. The analysis of this generic bridge is presented in Chapter 5.

The third type is the Candir Bridge, a reinforced concrete T-Girder bridge on a state highway in northwest Turkey. The actual superstructure plans were obtained from the GDH. The analysis of this bridge is presented in Chapter 6.

This selection was a representative sample of Turkish bridges as 90 percent of bridges in Turkey are concrete, although some are prestressed concrete and Gerber-type bridges (simply supporting the suspended segment of the center span on the cantilevered ends of the girders of the side spans). A prestressed concrete bridge was not selected for the analysis, as these bridges are relatively new and were designed to higher axle loads than the relatively old bridges considered in this study.

Live Load

The live load used in the analysis of the bridges was the HET shown in Figure 1. It has a total weight of 104.7 tonnes (230.8 kips) distributed over nine axles.

Figure 1. A heavy equipment transporter with an M-1 tank

To get the standard truck geometry of AASHTO-SSHB (AASHTO 1977; AASHTO 1996), the load on each axle was assumed to act at two points 1.83 m (6 ft) apart from each other in the transverse direction, as shown in Figure 2. Heavy Equipment Transporter (HET) distributes the load over 28 tires; however, it was assumed that the load was distributed over 18 tires. Results of the analysis were conservative with this assumption as in the actual case the total load is spread on 28 tires. This figure also shows the axle spacing and axle load configuration for the HET.

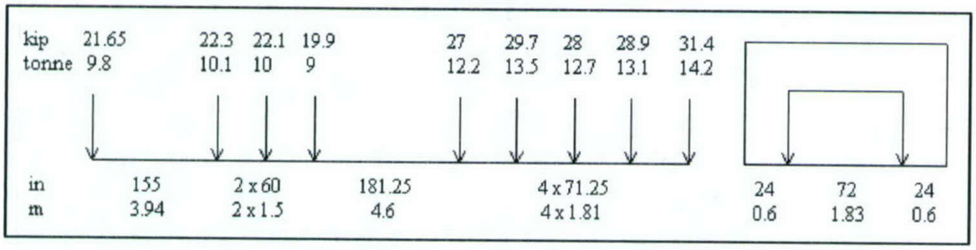


Figure 2. Axle loading and assumed transverse geometry of HET for analysis

Software

Structural Analysis Program 2000 (SAP2000) was used to model and analyze the bridges. Structural Analysis Program (SAP) is a finite analysis program that was initially developed at the University of California-Berkeley about 25 years ago. SAP2000 is the latest release of the SAP series of computer programs, which has been used widely for structural analysis. The ongoing usage of the program and continuing program upgrades are strong indicators that most program bugs have been identified and corrected (Computers and Structures, Inc.

2003a). This program was selected as it is a well-known and accepted finite analysis program, and the investigators had prior knowledge of the program. A comprehensive series of verification examples are provided with the software (Computers and Structures, Inc. 2002b).

AutoCAD 2002, a computer aided drawing program, was used to prepare drawings. This program was developed by Autodesk Incorporated (AutoCad2002 2001).

PCACOL V3, a concrete analysis program, was used to obtain interaction diagrams and to analyze the sections of the members. This program was developed by the Portland Cement Association (Portland Cement Association 1999). Version 3 is the latest edition of PCACOL and was developed in 1999. This version is based on the 1995 edition of ACI 318 code. The basic analysis equations did not change after this edition.

Report Layout

An introduction, general remarks and objectives are given in Chapter 1. General background of transportation systems, specifically highway transportation, and design specifications in Turkey are presented in Chapter 2. Chapter 3 compares Turkish material and design specifications with the ones used in the United States (U.S.). Chapter 4 presents the analysis of an open spandrel arch bridge in Turkey. Chapter 5 presents the analysis of a concrete slab, steel I-Girder bridge. The analysis of a simple span reinforced concrete T-Girder bridge is presented in Chapter 6. Chapter 7 includes a comparison and discussion of the results. Additionally, an inventory card example of a bridge in Turkey is presented in Appendix A. Appendix B presents live load truck geometries of Turkish and American specifications. Computer output diagrams of the Birecik Bridge are presented in Appendix C. Appendix D and Appendix E present pictures of the Birecik and Candir Bridges, respectively. The bridge information sheets are given in Appendix F.

Disclaimer

This research study analyzed three selected Turkish bridges focusing on the live load, the dead load, and the impact only. Other possible loads were not considered. Assumptions were made in order to complete the analysis, when actual conditions were unavailable. If the design drawings were available, it was assumed that the bridges had been constructed perfectly according to the drawings. Considerable resources have been expended to complete the analyses and to assess the capacities of the bridges according to these assumptions. The results obtained from this research study *do not necessarily show the actual conditions of the bridges.* More accurate and dependable results could be obtained by conducting nondestructive and/or destructive tests, field investigations, and actual measurements of the bridges.

2 General Background

Turkish Transportation System

The Ministry of Transportation is responsible for Turkish railways, airports, and seaports. The General Directorate of Highways, a directorate of Ministry of Public Works and Settlement, is responsible for the highways and bridges.

Railways

Under the Ottomans, at the end of the 19th century non-Turkish companies constructed the portion of the Berlin-to-Baghdad railroad that crossed Turkey, as well as a few other lines used mostly for mining development and the export of agricultural products. During the first decades of the Turkish Republic, track length was increased from 4,018 km (2,497 mile) in 1923 to 7,324 km (4,551 mile) in 1950. As of today, the total length of the main railway network is 8,607 km (5,348 mile). Almost all railways are single-tracked (only one railway line) and nonelectrified. Although rail lines linked most important cities, there were few cross connections between lines, and routes were often circuitous. As a result of increased use of trucks, the railroads carry only one-quarter of surface freight, mostly long-haul bulk commodities (Metz 1996).

There are more than 24,000 bridges along Turkish Railways, 92 percent of which were constructed more than 40 years ago. Half of all bridges were constructed during the first twenty years of the Turkish Republic, between 1920-1940. For construction materials, Turkish Standards TS-500 and TS-708 (Turkish Standards Institution (2000); Turkish Standards Institution (1996)) are being used. The specification used for design and loading criteria is the German Specification for Design of Railways (Turkish State Railways 2003).

Airports

Turkey has 105 airports, 69 of which have paved runways, and 20 of which are international (Metz 1996). The main four international airports are located in Istanbul, Ankara, Izmir, and Antalya. In 2000, the total number of passengers carried to, from, or within Turkey on all airlines reached about 35 million.

Seaports

Turkey has a coastline of 8,333 km (5,178 mile). Istanbul, the most important port, is followed by Mersin, Izmir, Iskenderun, and Kocaeli. The major seaports are shown in Figure 3. Shipping is much less important than land transport, but its volume has expanded rapidly in the early 1990s. Other than the ferry across the Lake Van, internal shipping is insignificant because few rivers in Turkey are navigable (Metz 1996).

Roadways

After World War II, transportation development concentrated on the roadway network system (Metz 1996). As a result, by 2003 Turkey has nearly 63,219 km (39,283 mile) of all-weather highways, of which about 92.8 percent is paved. These roads can be classified as expressways, highways, and provincial roads controlled by the GDH Republic of Turkey (2003). There are also some 300,000 km (186,000 miles) of dirt roads in rural areas, which are controlled by the Ministry of Cultivation and Village Affairs. GDH is not responsible for the construction or maintenance of these roads. The main expressway is the one connecting Europe-Istanbul-Ankara. A map showing the expressway network is given in Figure 3.

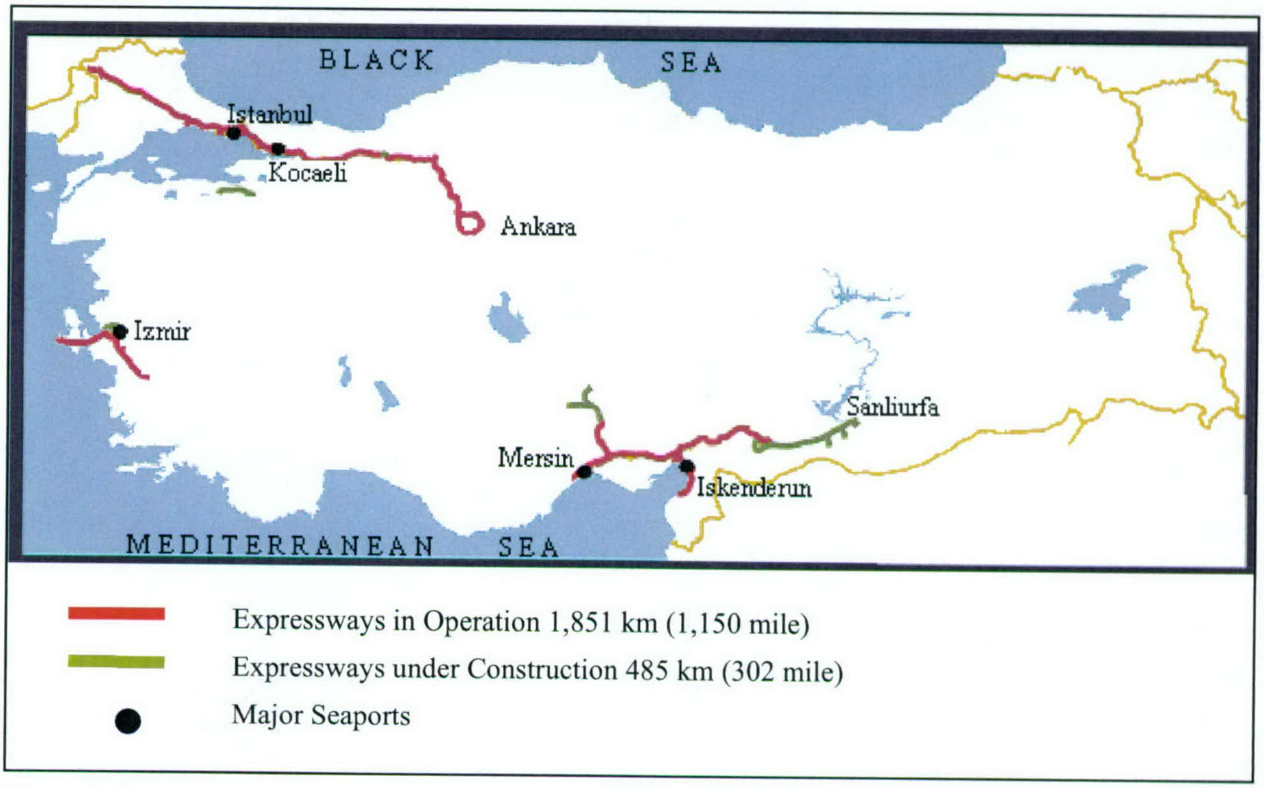


Figure 3. Turkish expressways and major seaports, 2000 (GDH 2001)

The highways connect major cities while provincial roads connect local cities with highways. Table 1 shows the roadway network according to surface type (Republic of Turkey 2003).

Table 1
By 2003, the Length of the Roadway Network According to Surface Type (Republic of Turkey 2003)

		Asphaltic Concrete	Bitumen Surfacing	Stabilized Gravel	Other	Total
Expressways	km	1,851	0	0	0	1,851
	mile	1,150	0	0	0	1,150
Highways	km	6,082	24,669	302	265	31,318
	mile	3,779	15,328	188	165	19,460
Provincial Roads	km	795	25,274	2,303	1,678	30,050
	mile	494	15,704	1,431	1,043	18,672
Total	km	8,728	49,943	2,605	1,943	63,219
	mile	5,424	31,033	1,619	1,207	39,282

In the early 1980s, Turkey began a major project to develop highways that would traverse the country making it possible for Turkey to handle increased levels of freight between Europe and the Middle East. Truck transport of surface freight increased from about 25 percent of the total freight in 1950 to more than 95 percent by the end of 2002. Passenger cars make up 61.5 percent and trucks make up 17 percent of the total number of registered vehicles in 2002 of 7.5 million (Republic of Turkey 2003). Traffic volume and axle load surveys are carried out regularly by the GDH. Truck traffic volume is higher on the roads connecting major ports to the cities and on the Trans-European Motorway (or Expressway). Traffic flow map of state highways in 2000 is given in Figure 4 (GDH 2001).

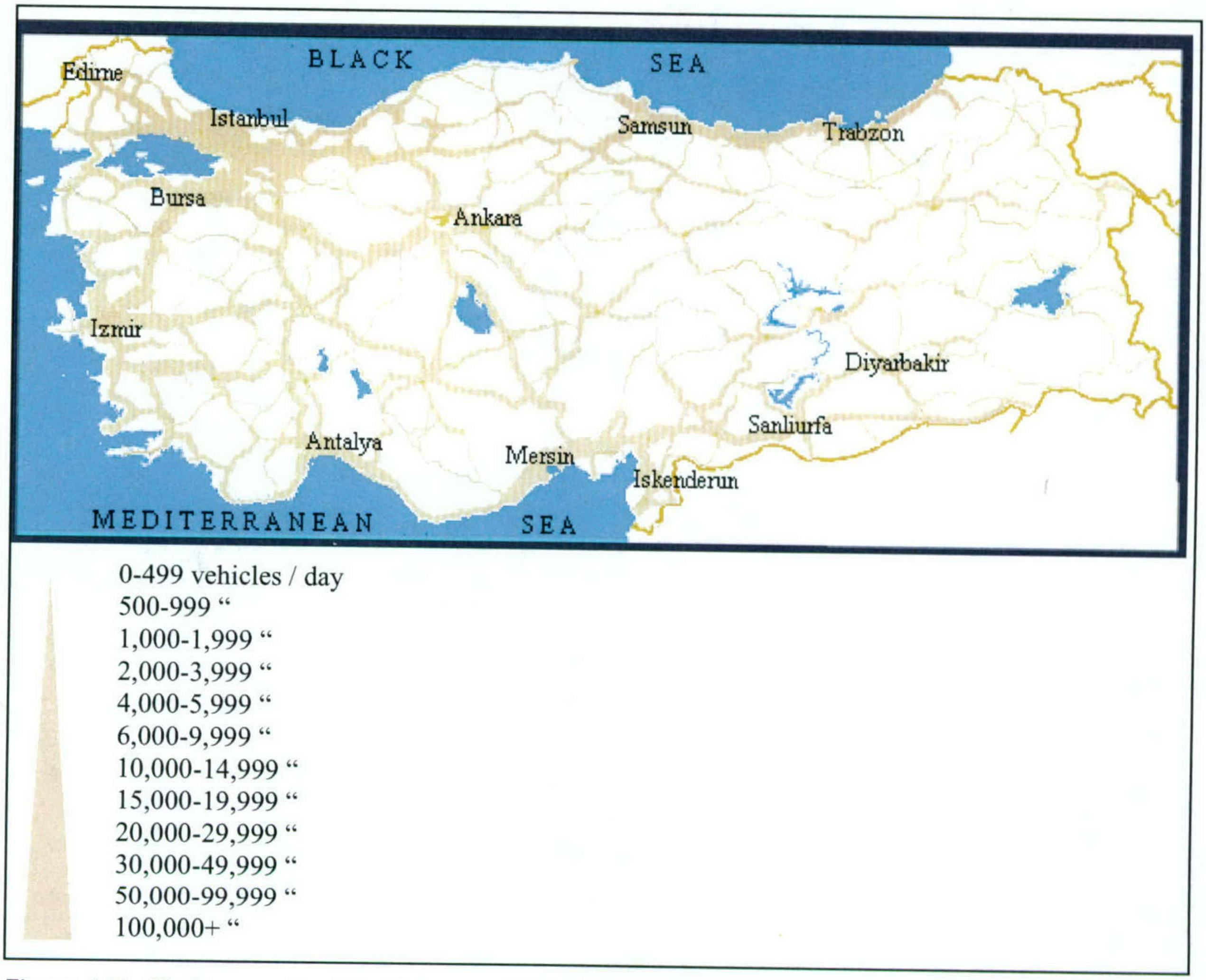


Figure 4. Traffic flow map in state highways, 2000 (GDH 2001)

Transit traffic on the Trans-European Motorway, which connects Europe to cities on the Persian Gulf, was disrupted by the 1990 Iraqi invasion of Kuwait and the resulting UN embargo (Metz 1996). Transit traffic volume is expected to increase after the change in the Iraqi regime. The Trans-European Motorway is shown in Figure 5 (GDH 2001).

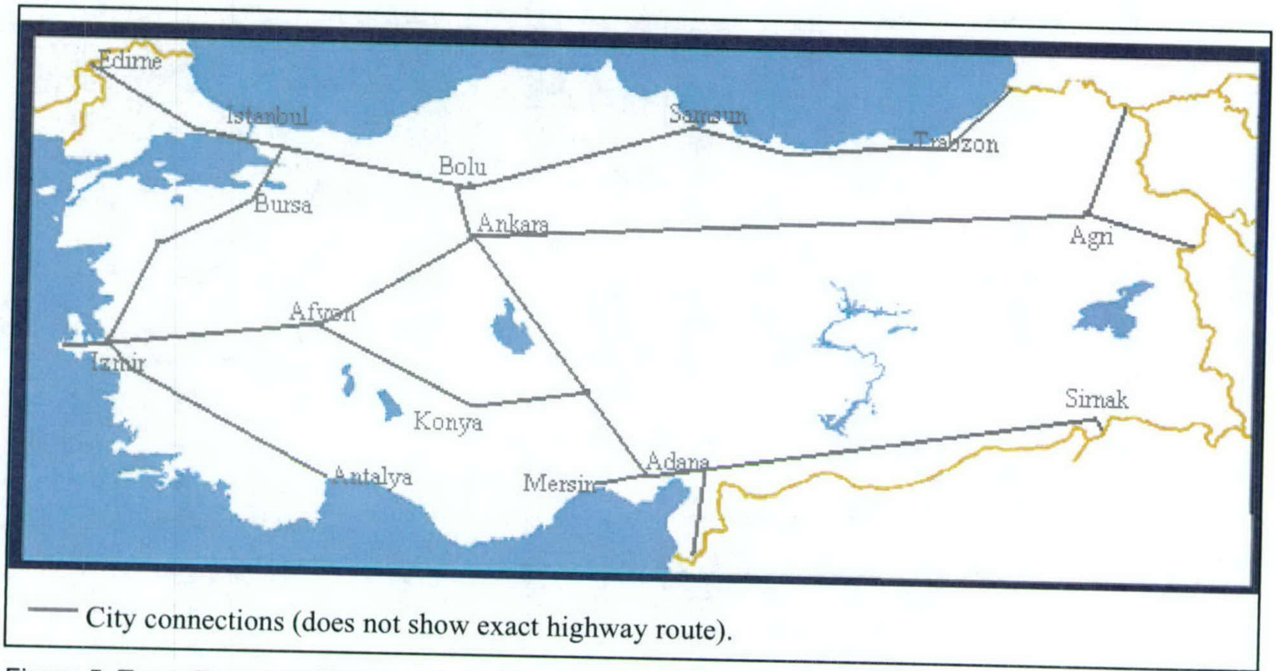


Figure 5. Trans European Motorway in Turkey, 2000 (GDH 2001)

General Directorate of Highways

The GDH, the headquarter of which is situated in Ankara, consists of 17 regional divisions, 116 district offices, 1 equipment and supply office, and one central workshop. Duties and responsibilities of the various departments are clearly defined and all of the activities are coordinated from the headquarters of GDH. These 17 divisions are spread throughout the country and each assists with the work in its region. Except for the 17th division, all divisions are responsible for all road types within their regions. The 17th division, based in Istanbul, is only responsible for the administration of expressways (Republic of Turkey 2003).

In the GDH there are 23,837 personnel, of whom 2,588 are technical personnel and 408 of these are located at the General Directorate in Ankara (Republic of Turkey 2003). The organizational chart of the GDH is given in Figure 6.

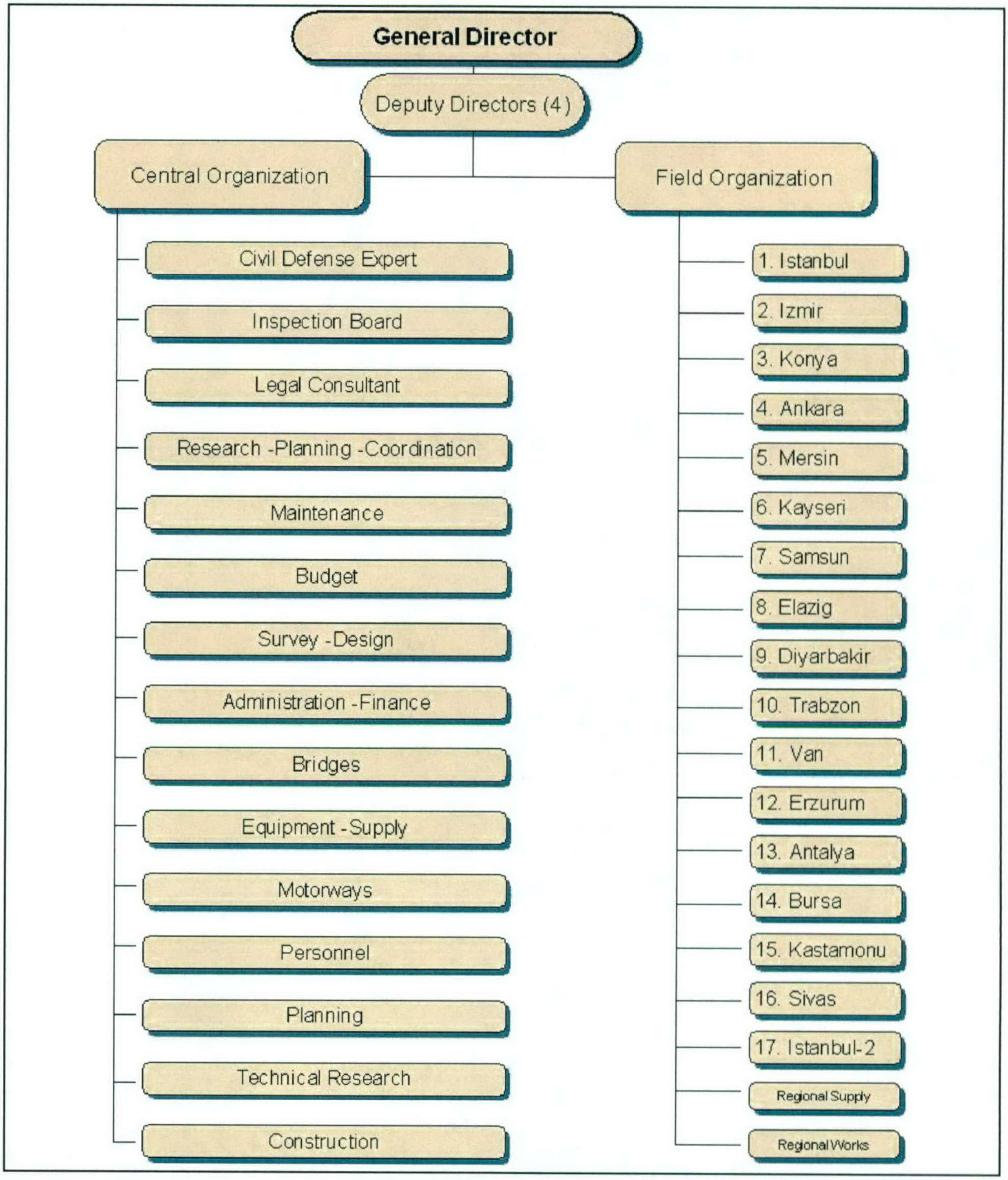


Figure 6. Organization chart of the General Directorate of Highways (Republic of Turkey 2003)

The GDH is a public institution, which is funded primarily through the general budget of Turkish Government, but also has supplementary budget contributions. In each fiscal year, the Parliament allocates the general budget for each department in accordance with the policies of government and the investment plans. Each year, the GDH collates all the projected expenditure requirements from the 17 regional divisions for necessity and emergency evaluation. The budget is apportioned according to this evaluation (Republic of Turkey 2003).

Due to the present economic crisis in Turkey, the GDH is facing budgetary problems. Due to inadequate allocations from the general budget, external resources from international finance institutions, such as the World Bank and European Investment Bank, are used for the construction of feasible bridges (Republic of Turkey 2003).

Another preferred type of funding is private financing via the Built-Operate-Transfer (BOT) model (Republic of Turkey 2003). In the BOT model, local, or foreign companies finance and construct bridges using their own resources. They then operate the bridges for a certain period of time, to pay of the debt, repay the equity, and then transfer the bridge to the government at the end of a concession period at no cost to the government (Menheere, Sebastiaan, and Spiro 1996).

Present Condition of Bridges

By the end of 1950 there were 1,028 bridges with a total length of 34 kilometers (21 miles) in Turkey. The Table 2 shows the number and total length of the existing bridges according to their types by 2001 (Republic of Turkey 2003).

Table 2
The Number and the Length of the Turkish Bridges According to Their Types, 2001 (Republic of Turkey 2003)

		Concrete	Stone	Composite	Steel	Total
Number		4,370	120	320	36	4,850
Length	km	167	6	13	2	188
	mile	103.8	3.7	8.1	1.2	116.8

The Turkish Bridge Maintenance Division was established in 1964. The system used for the inventory of existing bridges is working, although most of the data is outdated. All of the bridges are under the responsibility of the GDH, and are documented. The inventory data system is a card system in which each bridge has its own card and the data about that bridge is entered on the card. These cards exclude details, such as the bridge components and construction and maintenance history. The information contained on the cards includes type of bridge, design materials, design load, geometric features, design drawings, and some pictures of the bridge (Turkish State Railways 2003). The cards are available at the GDH in Ankara. A copy of existing bridge inventory card for the Candir Bridge is given in Appendix A.

There is no regular inspection system for bridges in Turkey. The only inspections that have been conducted are a result of failures reported by drivers after accidents or natural disasters. The main reason for a weak inspection system and outdated inventory data is insufficient budget allocation by the government. The Bridge Maintenance Division is unable to do even the basic inspection and maintenance work due to its tight budget (Japan International Cooperation Agency 1996). This situation has gotten worse since 1996 because of the impact of the earthquakes in 1999 and the subsequent economical crisis in 2001.

Material - Manufacturing Specifications

Materials and the methods for testing of the materials used in bridge construction are in accordance with Turkish Standards (TS). Established in 1960, Turkish Standards Institution (TSI) has published a considerable number of standards covering the use, manufacturing, and testing of materials. The development of TS has been primarily adapted from publications of the American Association of State Highway Officials (AASHO), and American Society for Testing and Materials (ASTM). To enable Turkish products to be exported to the other countries, such as European countries, the materials produced in Turkey according to TS are also in compliance with the standards of other countries, such as European Standards, and Euro Norm (Japan International Cooperation Agency 1996).

Most of the construction materials, such as cement, steel rebars, and prestressing strands, are produced in Turkey. Some manufacturers have International Standards Organization 9001 (ISO 9001) certification. Ready-mix concrete, produced according to Turkish Standards, is available for construction. In recent years, there has been an increase in the construction of precast and prestressed concrete bridges. Fabricated steel bridges are not very popular due to the higher capital costs associated with steel construction and the high costs related to the maintenance of steel bridges. Facilities for fabrication of steel bridges are available (Japan International Cooperation Agency 1996).

Design Specifications

The GDH is responsible for the design of all State Highway and Expressway bridges except for those, which are funded by the World Bank. Projects funded by the World Bank are based on selection criteria of a consulting engineering firm, identified by the rules of the Bank. The Technical Specifications for Road Bridges[1] (TSRB) GDH 1982), published (in Turkish) in 1982, is the main specification for design of highway bridges. TSRB was adopted from AASHTO (1977). It includes requirements for the design loading, load distribution, and allowable limits for the various types of construction materials such as timber, concrete, and steel. If the bridge is in a seismic zone, the GDH requires the use of AASHTO-Standard Specifications for Seismic Design of Highway Bridges (AASHTO 2003). If it is a special bridge or the span is 'very long' (although the

[1] Technical Specification for Roadway Bridges (TSRB) (GDH 1982), hereafter in this report, reference will only be made to TSRB.

limit is not defined), it is required to be designed to a foreign specification decided by the GDH (Japan International Cooperation Agency 1996). For most of the bridges, the TSRB is only used for the loading and geometric criteria while AASHTO (1977)/AASHTO (1996) is used for all other requirements (Japan International Cooperation Agency 1996).

Engineering Works Criteria Report (EWCR), which was published (in Turkish) by the GDH in 1997, is a reference that gives general criteria and requirements for the construction of bridges. This report requires that "Unless otherwise stated, all bridges shall be designed according to the latest edition of AASHTO (1996) or AASHTO (1977)" (GDH 1997). This report also covers the properties of construction materials, such as concrete, steel, and prestressing strands.

Standard Bridge Types, published (in Turkish) by the GDH in 1953, is a reference for the design of reinforced concrete bridges. The concrete used in these bridges was required to have compression strength of 22.1 MPa (3,200 psi). The steel type was ST37. The live load for the design was HS20. It only covered simple span T-Girder bridges. It included all the reinforcement and formwork details for both the substructure and the superstructure for which maximum span length was 15.70 m (51.5 ft) (GDH 1953c).

3 Comparison of Turkish and American Specifications

Materials

The properties of materials to be used in construction of bridges are defined in the TS, published by TSI. If a material is not found in the TS, the GDH recommends the use of a relevant foreign standard (Japan International Cooperation Agency 1996). The design values for concrete, steel, and prestressing strands in the TS are compared to the values used in the United States in the following subsections.

Specified Concrete Properties

The concrete used in Turkish bridges until the 1980s had a 28-day compressive strength of 22.1 MPa (3,200 psi) (GDH 1953c). As ready mixed concrete was not common during the construction of older bridges, the compressive strength of concrete prepared at the site would have varied for these bridges.

Table 3 shows the concrete types currently used, and the corresponding specified compressive strengths for different applications in Turkey (GDH 1997). Almost all of the concrete used by the construction industry today is ready mixed concrete, which is much more reliable and consistent than the concrete mixed at the construction sites in older bridge construction.

Table 3
The Compressive Strength of Concrete Currently Used in Turkey (GDH 1997)

Type of Application	Designation*	fc'**	
		MPa	psi
Reinforced Concrete	C25	25	3,630
Post-Tensioned Prestressed Concrete	C35	35	5,080
Precast Prestressed Concrete	C40	40	5,800

* C stands for concrete and the number following C represents the specified compressive strength of the concrete in Mega Pascal after 28 days.
** Compressive strength of concrete at 28 days.

As shown in Table 3, the 28-day compressive strength of reinforced concrete is typically 25 MPa (3,600 psi), whereas the compressive strength of prestressed

concrete is on the order of 35-40 MPa (5,000-6,000 psi), which is on the order of the average strength of the prestressed concrete used in bridges in the United States.

Steel

ST37 type steel was used in Turkish bridges until the 1980s. The tensile strength and yield limit of ST37 steel is given in the Turkish Standard-648 (TSI 1980). In the TSRB, published in 1982, only ST37 steel is described. Today S420 steel, which is defined in the Turkish Standard-500, is being used (TSI 1996). The S420 is produced as deformed rebars with diameters of 6, 8, 10, 12, 14, 16, 18, 20, 22, 24, 25, 26, 28, 30, 32, 40, and 50 mm (0.24, 0.31, 0.39, 0.47, 0.55, 0.63, 0.71, 0.79, 0.87, 0.94, 0.98, 1.02, 1.10, 1.18, 1.26, 1.57, 1.97 in, respectively) (TSI 1996).

According to AASHTO (1996), Grade 40 and Grade 60 type steels are currently used in concrete bridges in the United States. The properties of Grade 40 and Grade 60 steels are defined in the standards published by ASTM A617 (ASTM 1996). The major material properties of these steels are summarized in Table 4.

Table 4
Properties of Steel Used in Turkey and in the United States (AASHTO 1996, TSI 1980, and TSI 2000)

		AASHTO (1996)		TSRB	
		Grade 40	Grade 60	ST37	S420
f_y†	MPa	275.8	413.7	235.1	420
	ksi	40	60	34	61
f_s‡	MPa	137.9	165.5	137.9	165.5
	ksi	20	24	20	24
E*	MPa	200,000		200,000	
	ksi	29,000		29,000	

† Minimum yield limit
‡ Allowable stress
* Modulus of elasticity

As seen in Table 4, the same modulus of elasticity is used in both specifications. Yield limit of S420 steel is 61 ksi, which is almost the same as the yield limit of Grade 60 steel. Allowable stresses for Grade 60 and S420 are equal. Although allowable stresses for ST37 steel and Grade 40 steel are equal, the yield limit of Grade 40 steel is about 15 percent higher than that of ST37 steel.

Prestressing Strand

For prestressed concrete bridges, Low Relaxation 7-strand Grade 270, which has a minimum yield limit of 1,861 MPa (270 ksi), is being used in Turkey (GDH 1997). The nominal diameters of these strands are 12.7 mm (0.5 in.) or

15.2 mm (0.6 in.) (GDH 1997). The same strand had been used before 1997. The properties of this strand are defined in ASTM A416 (ASTM 2002).

Loading

In both AASHTO (1996) and TSRB, structures are designed to carry dead load, live load, impact of the live load, and wind loads. Bridges are also designed for longitudinal forces, centrifugal forces, thermal forces, earth pressure, buoyancy, shrinkage stresses, rib shortening, erection stresses, ice and current pressure, and earthquake stresses, when they exist.

Dead Load

Table 5 shows the comparison of the weights of the main materials to be used in computing the dead load of a structure according to AASHTO (1996) and TSRB. As shown in Table 5, unit weights of steel and aluminum are the same in both standards. Unit weight of cast iron is slightly different due to conversion from U.S. customary units to International System of Units (SI). AASHTO (1996) gives the unit weight of wood as 801 kg/m^3 (50 pcf); however, in TSRB it varies between 650-1,000 kg/m^3 (41-62 pcf), dependent on type of wood. Unit weight of concrete, which is constant in AASHTO (1996), also depends on the type of concrete and changes between 2,300-2,500 kg/m^3 (143-156 pcf) in the TSRB. Unit weight of gravel is lower by 17 percent in TSRB than it is in AASHTO (1996). Unit weight of asphalt plank is 15 percent higher in TSRB than it is in AASHTO (1996). It is clear that there are very slight differences in two specifications regarding the unit weight of the major construction materials.

Table 5
Comparison of Unit Weight of Materials for Dead Load Calculation (AASHTO 1977; GDH 1982)

	AASHTO (1996)		TSRB	
	kg/m^3	pcf	kg/m^3	pcf
Cast Steel	7,850	490	7,850	490
Cast Iron	7,208	450	7,250	452
Aluminum Alloys	2,800	175	2,800	175
Timber	801	50	650-1,000(1)	41-62(1)
Concrete	2,403	150	2,300-2,500(2)	143-156(2)
Compacted Sand-Earth-Gravel	1,922	120	1,600	100
Asphalt Plank	1,730	108	2,000	124

(1) Dependent on the type of wood.
(2) Dependent on the concrete type, plain or reinforced concrete.

Live Load

In AASHTO (1996) and TSRB, live load is defined as the weight of the applied moving load of vehicles and pedestrians.

Standard Truck and Lane Loads

This article is translated into Turkish from AASHTO (1977). Therefore, the requirements and definitions of standard truck and lane load are identical in both specifications. The only difference in live loads in the two standards is what the number after H or HS represents. In AASHTO (1996), HS is followed by a number indicating the gross weight in *English (short) tons* (2,000 lb) of the tractor truck. On the other hand the same number in TSRB indicates the gross weight in *metric tons* (1,000 kg or 2,200 lb) of the standard truck. This is similar for the H truck and lane loading; therefore, any load in TSRB is 10 percent heavier than its equivalent in AASHTO (1996).

The geometry of standard trucks, which is slightly different due to the conversion from U.S. customary units to the SI units, is shown in Appendix B. Designation of loading classes are tabulated and compared in Table 6. The designations 1 and 2 are two namings of the same load.

Table 6
Comparison of Standard Truck and Lane Loadings

AASHTO (1996)		TSRB	
Designation 1	**Designation 2**	**Designation 1**	**Designation 2**
H15 H20 H15-S12 H20-S16	H15-44 H20-44 HS15-44 HS20-44	H10 H15-S12 H20-S16 H30-S24	H10 HS15 HS20 HS30
The number after HS indicates the gross weight in <u>(short) tons</u>. The number '44' stands for 1944 edition of AASHTO (1996).		The number after HS indicates the gross weight in <u>metric tons</u>. 1 metric ton = 1.1 (short) ton	
The HS20 in TSRB is 10 percent heavier than the HS20-44. The HS30 in TSRB is 65 percent heavier than the HS20-44.			

As shown in Table 6, TSRB includes an HS30 load, which is 65 percent heavier than the AASHTO (1996) HS20-44 load. In TSRB, all of the trucks except the H10 are HS-Type trucks. Therefore only HS-Type trucks are used in practice for design of bridges.

Traffic Lanes

In AASHTO (1996) the lane loading or standard truck is assumed to occupy a width of 3.05 m (10 ft). In TSRB it is 3 m (9.85 ft). The corresponding traffic lanes for roadway widths are shown in Table 7.

Table 7 Roadway Width and Corresponding Number of Traffic Lanes				
Width of roadway				Number of lanes
AASHTO (1996)		TSRB		
ft	m	ft	m	
20-30	6-9	20-30	6-9	2
30-42	9-12.8	30-43	9-13	3
42-54	12.8-16	43-54	13-16	4

Minimum Loading

In AASHTO (1996), the minimum loading for an interstate highway bridge is HS20-44 or an Alternate Military Loading (AML) of two-axle, 4 ft apart with each axle weighing 10,900 kg (24,000 lb), whichever produces the greatest stress.

In TSRB, the minimum loading is HS20, which is 10 percent heavier than HS20-44. In the past, some of the old bridges on secondary highways in Turkey were designed for HS15, which is 10 percent heavier than HS15-44 loading. In the U.S. the minimum loading for the bridges on secondary highways is given as HS15-44 in AASHTO (1977).

For expressways in Turkey, the minimum loading requirement is HS30 loading or an AML of two-axle, 4 ft apart with each axle weighing 18,100 kg (40,000 lb), whichever produces the greatest stress.

Minimum loading requirements are tabulated and compared in Table 8. As seen in this table, the minimum loads are much higher in the TSRB than those in AASHTO (1996). It is 65 percent heavier for interstate highways (expressways in Turkey) and 45 percent heavier for secondary highways.

Table 8 Minimum Loading Requirements		
	AASHTO (1996)	TSRB
Interstate / Expressway	HS20-44 or AML*	HS30† or AML*
Secondary Highways	HS15-44	HS20‡
† HS30 in TSRB is 65 percent heavier than HS20-44 in AASHTO (1996). ‡ HS20 in TSRB is 45 percent heavier than HS15-44 in AASHTO (1996). * Alternative Military Loading		

Application of Live Load

This article is translated into Turkish from Article 1.2.8 in AASHTO (1977) (AASHTO (1996), Article 3.11). In TSRB traffic lane design width, which is 3.05 m (10 ft) in AASHTO (1996), is given as 3.0 m (9.85 ft). Application of lane loads and truck loads on continuous spans and loading for maximum stress requirements are identical in both standards.

Impact

In both AASHTO (1996) and TSRB, impact is to be applied to:

a. Superstructure, including legs of rigid frames.

b. Piers excluding footing and those portions below the ground line.

c. Concrete or steel pile portions above the ground line that support the superstructure.

In both standards impact is not to be applied to:

a. Abutments, retaining walls piles except as specified above.

b. Foundation pressures and footings.

c. Timber structures.

d. Sidewalk loads.

e. Culverts and structures having 3 ft or more cover (in TSRB, it is 1 m or 3.28 ft)

As seen in Table 9, there is a slight difference in impact formulas used in the specifications. This difference is due to conversion from U.S. customary units to SI units.

Table 9
Comparison of Impact Formulas

AASHTO (1996)		TSRB	
m	ft	m	ft
$I = \frac{15.24}{L+38}$	$I = \frac{50}{L+125}$	$I = \frac{15}{L+37}$	$I = \frac{49.2}{L+121.4}$

I: Impact fraction (maximum 30 percent in both standards).
L: Length of the span that is loaded to produce the maximum stress in the member.

Impact for culverts, as seen in Table 10, impact factor for design of culverts in AASHTO (1996) is 10, 20, or 30 percent. The maximum impact factor in TSRB is 30 percent, the same as in AASHTO (1996). The TSRB defines the impact factor as linearly proportional to the cover depth.

Table 10 Comparison of Impact Fraction for Design of Culverts			
AASHTO (1996)			**TSRB**
Culvert cover		**Impact**	**Culverts up to 1 m (3.28 ft) cover**
ft	**m**	**Percent**	Impact fraction is linearly proportional to the cover depth and changes between 0 to 30 percent.
0-1 1-2 2-3	0-0.3 0.3-0.6 0.6-0.9	30 20 10	
I= Impact fraction (maximum 30 percent in both standards)			

Reduction in Load Intensity

According to AASHTO (1996), in any member where maximum stresses are produced by loading a number of traffic lanes simultaneously, a percentage of the live loads is used in view of the improbability of coincident maximum loading.

As shown in Table 11, the percentages of live load to be used in design are identical in both AASHTO (1996) and TSRB. For one or more lanes, 100 percent of the live load is used in the calculations. For three lanes and for four or more lanes, 90 and 75 percent of the live load is used in the calculations, respectively.

Table 11 Comparison of Impact Fraction for Culverts		
	AASHTO (1996)	**TSRB**
	percent	**percent**
One or two lanes	100	100
Three lanes	90	90
Four lanes or more	75	75

Overloading

a. AASHTO (1996). In the 1977 edition of AASHTO (1977), it is required for all loadings, except the H20 and HS20, that the truck load (H or HS) is to be increased by 100 percent, and without concurrent loading of any other lanes. Combined dead, live, and impact stresses resulting from such loading cannot be greater than 150 percent of the allowable stress.

AASHTO (1996) does not define an infrequent heavy load, but requires a provision made for overloading (AASHTO (1996), Article 3.5.1). It also allows the operating agency to determine the percent increase of allowable stresses to be used (AASHTO (1996), Table 3.22.1A).

The overloading provision in AASHTO (1996) is only for bridges designed for loading less than the H20, which is not applicable today. Therefore, most states have made their own provisions for overloading and design their bridges according to some infrequent heavy vehicles defined in their local specifications.

b. TSRB. According to TSRB 1982, any lane on a highway bridge is to be loaded with the heavy commercial hauler, a diagram of which is shown in Figure 7. In practice, this provision has not been followed. According to Ms. Fatma Sahin, an engineer in the Bridge Design Division of the GDH in Ankara, no highway bridges have been designed or checked for this heavy commercial hauler (GDH 2003).

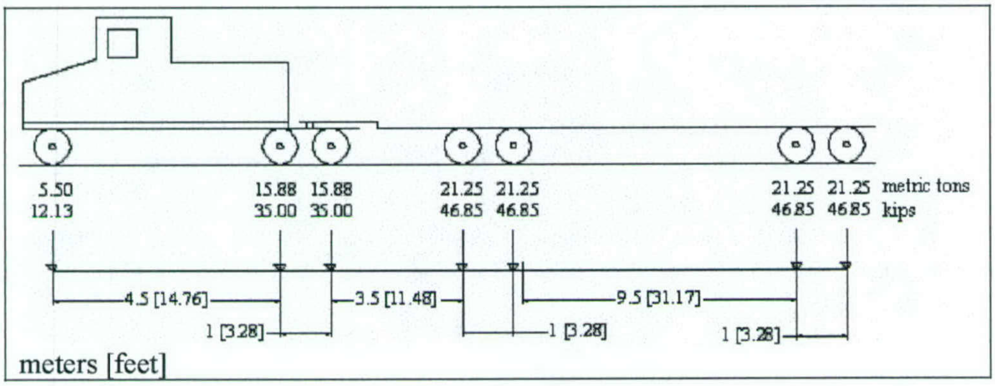


Figure 7. Heavy commercial hauler of the TSRB (GDH 2003)

Expressway bridges are overloaded for two types of loading, without concurrent loading of any other lanes. Type-A loading is a column of military tanks, a diagram of which is given in Appendix B. Each tank weighs 136,000 kg (300 kips) and the column has a 30.5 m (100 ft) spacing between tanks. Type-B loading is a series of heavy commercial haulers, a diagram of which is given in Appendix B. Each vehicle weighs 154,000 kg (340 kips) and the vehicles are lined with 30.5 m (100 ft) spacing. As in AASHTO (1996), combined dead, live, and impact stresses resulting from such loadings are limited to no more than 150 percent of the allowable stresses.

The following formula is used to calculate the impact for overloading. The calculated impact cannot be greater than 0.1 for tracked vehicles and 0.25 for tired vehicles. In this formula, 'L' is the span length in meters and 'H' is the thickness of the cover.

$$I = 0.4 - 0.0008 \cdot L - \frac{H}{10}$$

Sidewalk Loading

a. AASHTO (1996). Sidewalk floors, stringers, and their immediate supports are designed for 4.07 kPa (85 psf). Girders, trusses, arches, and other members are designed for the following live loads:

Spans of 0-25 ft (0-7.6 m)	4.07 kPa (85 psf)
Spans of 25-100 ft (7.6-30.5 m)	2.87 kPa (60 psf)

Spans over 100 ft (30.5 m) according to the formula,

$$P = \left(30 + \frac{3,000}{L}\right)\left(\frac{55 - W}{50}\right)$$

P: Live load per square foot (P<60).

L: Loaded length of sidewalk in feet.

W: Width of sidewalk in feet.

b. TSRB. For the sidewalks of highway bridges, live load is taken to be 2.94 kPa (61.5 psf). Curbs or sidewalks, which are wider than 0.6 m (1.97 ft), are considered for this loading. The sidewalk loading is summarized and compared in Table 12.

Table 12
Sidewalk Uniform Loading Comparison

Span Length		AASHTO (1996)		TSRB	
ft	m	Pa	psf	Pa	psf
0-25	0-7.6	4,070	85	2,942	61.5
25-100	7.6-30.5	2,873	60	2,942	61.5

As seen in Table 12, sidewalk loading is 2.94 kPa (61.5 psf) in TSRB and is not dependent on the span length. For a typical span length, the specifications require almost the same sidewalk loading.

Distribution of Loads

The load distribution method in TSRB is similar to the method explained in AASHTO (1996). In TSRB, "The Distribution of Loads" chapter was directly translated from AASHTO (1977). The distribution factors used to find the bending moment in interior stringers and beams are given in a table in AASHTO (1996). The numbers in this table were converted into SI system and the same table is given in TSRB. Table 13 and Table 14 show the distribution factors for interior stringers for one lane and for two or more lanes, respectively.

Table 13
Distribution Factors for Bending Moment in Interior Stringers for One Lane

Type of Deck	AASHTO (1996)		TSRB	
	ft	m	ft	m
Timber Plank	S / 4.0	S / 1.22	S / 3.94	S / 1.20
Strip 4 in. (10 cm) thick or multiple layer floors over 5 in. (12.5 cm) thick	S / 4.5	S / 1.37	S / 4.43	S / 1.35
Strip 6 in. (15 cm) or more thick	S / 5.0	S / 1.52	S / 4.93	S / 1.50
Concrete On steel I-Girder Stringers and Prestressed Concrete Girders	S / 7.0	S / 2.13	S / 6.89	S / 2.10
On Concrete T-Girder	S / 6.5	S / 1.98	S / 6.56	S / 2.00
On Timber Stringers	S / 6.0	S / 1.83	S / 5.91	S / 1.80
On Concrete Box Girders	S / 8.0	S / 2.44	S / 7.87	S / 2.40
Steel Grid Less than 4 in. (10 cm) thick	S / 4.5	S / 1.37	S / 4.43	S / 1.35
Thickness of 4 in. (10 cm) or more	S / 6.0	S / 1.83	S / 5.91	S / 1.80

S: Average stringer spacing.

Table 14
Distribution Factors for the Moment in Interior Stringers for Two or More Lanes

Type of Deck	AASHTO (1996)		TSRB	
	ft	m	ft	m
Timber Plank	S / 3.75	S / 1.14	S / 3.77	S / 1.15
Strip 4 in. (10 cm) thick or multiple layer floors over 5 in. (12.5 cm) thick	S / 4.0	S / 1.22	S / 3.94	S / 1.20
Strip 6 in. (15 cm) or more thick	S / 4.5	S / 1.30	S / 4.27	S / 1.30
Concrete On steel I-Girder Stringers and Prestressed Concrete Girders	S / 5.5	S / 1.68	S / 5.41	S / 1.65
On Concrete T-Girder	S / 6.0	S / 1.83	S / 5.91	S / 1.80
On Timber Stringers	S / 5.0	S / 1.52	S / 4.93	S / 1.50
On Concrete Box Girders	S / 7.0	S / 2.13	S / 6.89	S / 2.10
Steel Grid Less than 4 in. (10 cm) thick	S / 4.0	S / 1.22	S / 3.94	S / 1.20
Thickness of 4 in. (10 cm) or more	S / 5.0	S / 1.52	S / 4.93	S / 1.50

S: Average stringer spacing.

Distribution factors used to find the bending moment in each transverse beam are adapted from AASHTO (1996) and given in TSRB in SI units. There can be small differences between numbers in the tables because of round-off errors resulting from conversion from U.S. customary units to SI units. Distribution factors for floor beams are compared and tabulated in Table 15.

Table 15 Distribution Factors for Bending Moments in Transverse Beams				
Type of Deck	AASHTO (1996)		TSRB	
	ft	m	ft	m
Plank	S / 4.0	S / 1.22	S / 3.94	S / 1.20
Strip 4 in. (10 cm) thick, wood block on 4 in. (10 cm) plank sub floor or multi-thickness plank over 5 in. (12.5 cm) thick	S / 4.5	S / 1.30	S / 4.43	S / 1.35
Strip 6 in. (15 cm) or more thick	S / 5.0	S / 1.52	S / 4.93	S / 1.50
Concrete	S / 6.0	S / 1.83	S / 5.91	S / 1.80
Steel grid less than 4 in. (10 cm)	S / 4.5	S / 1.30	S / 4.43	S / 1.35
Steel grid 4 in. (10 cm) or more	S / 6.0	S / 1.83	S / 5.91	S / 1.80
S: Spacing of floor beams				

Design Method

Before 1982, Turkish bridges were designed according to the ASD method. In the TSRB, only ASD is required for design of bridges. There was no requirement for the Strength Design Method, Load Factor Design (LFD), in the TSRB. Even though LFD is not defined in TSRB, for many years *"the bridges have been designed to AASHTO-SSHB using both ASD and LFD"* according to Ms. Fatma Sahin, an engineer in the Bridge Design Division of the GDH in Ankara (GDH 2003).

The latest edition of AASHTO-Load and Resistance Factor Design Specification (AASHTO 1998) is being translated into Turkish, and will be used by the GDH for the design of highway bridges in the near future. The Translation is being done by a local private company, Yuksel Construction Co. Inc., in Ankara (GDH 2003).

Allowable Stresses

For bridge design in Turkey, allowable stresses for both reinforced and prestressed concrete are obtained using the formulas given in AASHTO (1996). Allowable stresses for steel are tabulated in Table 4.

Load Factors for LFD

In the design of bridges in Turkey, the load factors used in LFD are same as the ones used in AASHTO (1996). For example, the formula applied to find the factored dead load and live load combination is $1.3 \cdot (DL + 1.67 \cdot (LL + I))$, where DL, LL, and I are dead load, live load and impact, respectively. This formula represents Group-I loading combination used in AASHTO (1996) for LFD.

Summary

The Turkish highway bridge design division currently uses the TSRB for the loading and geometric criteria only and AASHTO (1996) for all other requirements and design methods (Japan International Cooperation Association 1982). The latest AASHTO-LRFD (AASHTO 1998) is currently being translated into Turkish and will be used in the near future (GDH 2003).

The most significant difference between Turkish and American live loading is in what the number that follows the H or HS represents in the two standards. In TSRB, that number shows the weight of the truck in *metric tons (2,200 lb)*, but in AASHTO (1996) it represents the weight of the truck in *English (short) tons (2,000 lb)*. This also applies for lane loading. Therefore, *HS20 in TSRB is 10 and HS30 in TSRB is 65 percent heavier than HS20-44 specified in AASHTO (1996).*

Presently in Turkey the loading requirement for highway bridges is HS20 or HS30, depending on the importance of the highway (GDH 1997). For expressway bridges, it is required to design according to HS30 or AML, and to check the bridge for overloading with Type-A and Type-B vehicles, for which the diagrams are given in Appendix B.

4 Analysis of the Birecik Bridge

Introduction

The Birecik Bridge, which crosses the Euphrates River, was constructed in 1956. The bridge is in southeastern Turkey and is located on the state highway connecting Gaziantep and Birecik. The location of the bridge is marked with a star in Figure 8. Several photographs of the bridge were taken in February 2003 and are shown in Appendix D.

Figure 8. Location of the Birecik Bridge on the traffic flow map

The total length of the bridge, which consists of two parts, is 694.6 m (2,279 ft). The first part was constructed as a 15-span Gerber Girder reinforced concrete bridge. This part was not analyzed. The second part, which is over the river, is composed of five identical arches; each with a span length of 53.27 m (174.77 ft). A typical arch of the bridge is shown in Figure 9.

The Birecik Bridge was designed to HS20 of TSRB, which is 10 percent heavier than HS20-44 in AASHTO (1996) as described in Chapter 3. The external dimensions were obtained from AutoCAD (2002) drawings drawn by a Turkish architecture firm based on field surveys. Only the arch portion of the bridge was analyzed in this study. The bridge was analyzed according to AASHTO (1996) by modeling the bridge using SAP2000 using the external

dimensions of the members. SAP2000 is a finite analysis program as described in Chapter 1.

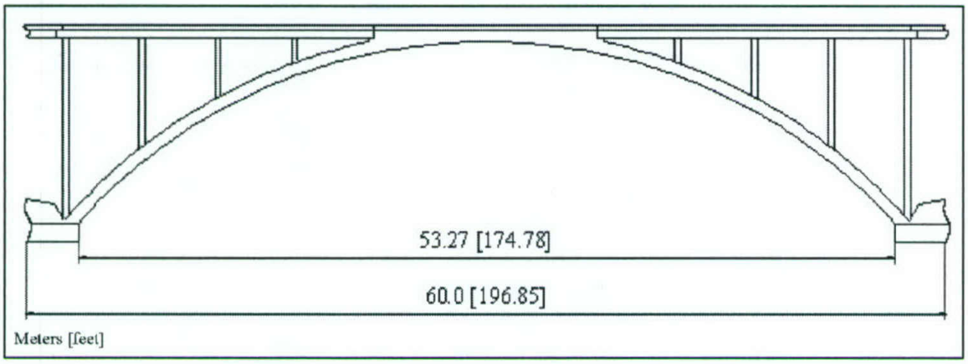


Figure 9. One of the arches of the Birecik Bridge

In the analysis, the steel used for the reinforcing bars is ST37, as described in Chapter 3. Compression strength of concrete, f_c' is 22.4 MPa (3,250 psi).

The load is transferred from the T-Girder concrete deck to the arch by means of columns placed at every 5 m (16.4 ft) on centers in the longitudinal direction. As shown in Figure 10, there are 3 rows of columns spaced at 2.65 m (8.69 ft) apart on centers in the transverse direction. The clear length of the highest column is 11.87 m (38.96 ft). The thickness of the arch rib is 0.8 m (2.63 ft). The clear spacing of the bridge, measured between the average water level and the bottom of the deck at the middle span, is 13.00 m (42.65 ft). The vertical dimensions of the bridge are given in Figure 11.

The curb-to-curb width of the Birecik Bridge is 8.50 m (27.9 ft), which makes it a two-lane bridge. The curb on each side has a width of 1.0 m (3.28 ft). The total width of the deck is 10.50 m (34.45 ft). The slab thickness is 30 cm (11.81 in.). The cross-section of the bridge deck is given in Figure 12.

Analysis of the Birecik Bridge

The external dimensions of the bridge were used in the model. Since no information was available relative to the reinforcement details, the bridge was analyzed assuming the minimum reinforcing steel allowed by the Building Code-Commentary published by ACI 318 (2002)/318R-02. The minimum reinforcement requirements of the Turkish standards are the same as the ones of the ACI.

Only the primary loads, dead load, live load and impact, were applied to the structure. As a live load, only one HET was assumed to be on the bridge at a time as prescribed in AASHTO (1996). The loads were not modified by any factor as the bridge was analyzed considering the service loads. Actual strengths of the sections were calculated without introducing any strength reduction factors.

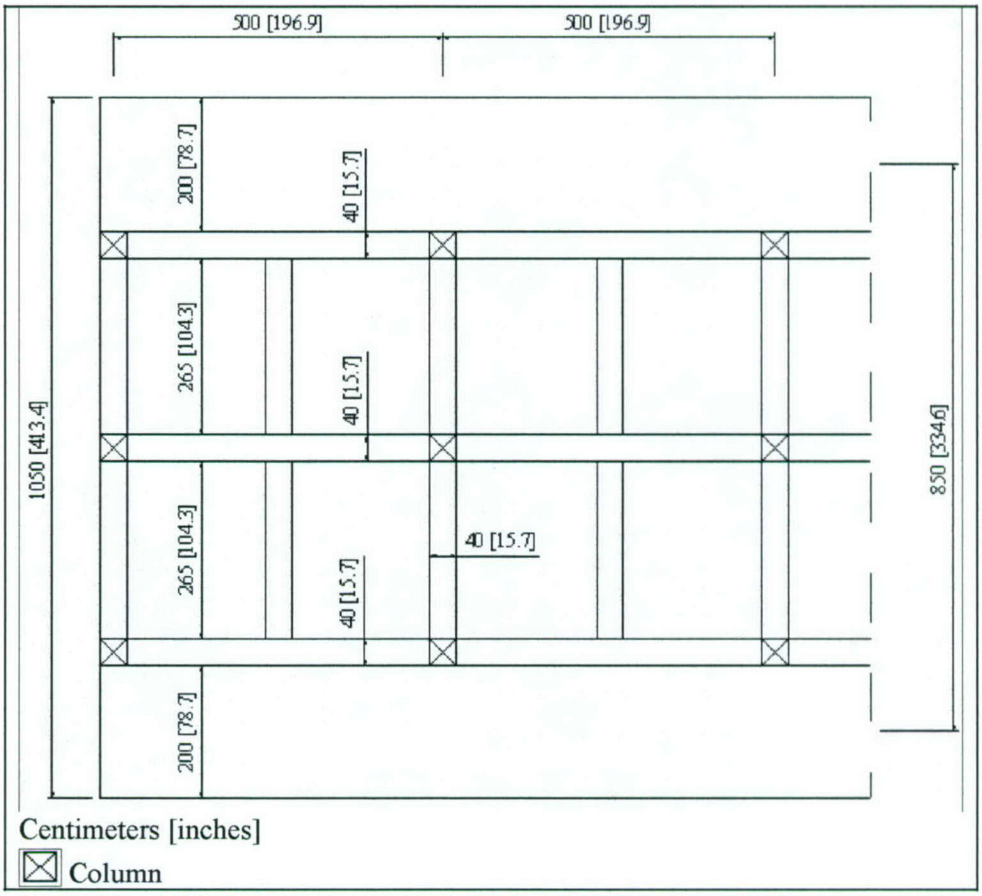


Figure 10. Plan of the bridge deck

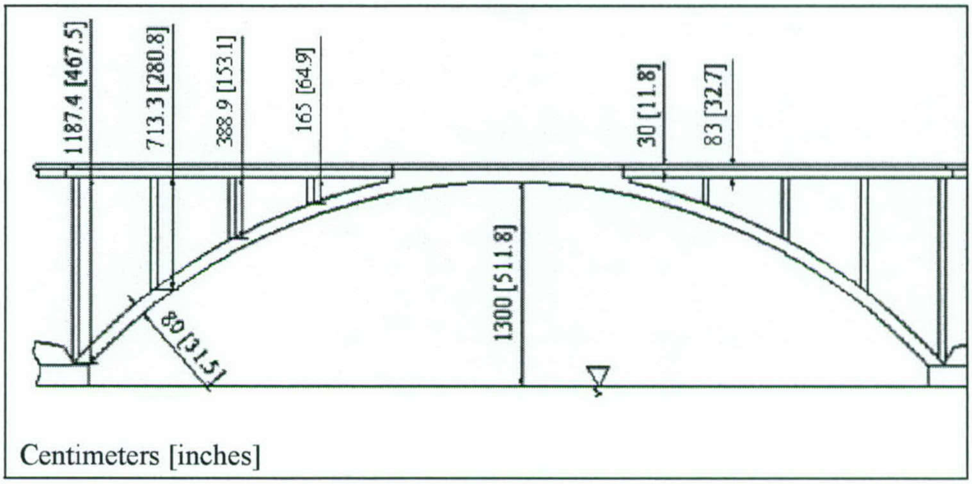


Figure 11. Vertical dimensions of the arch

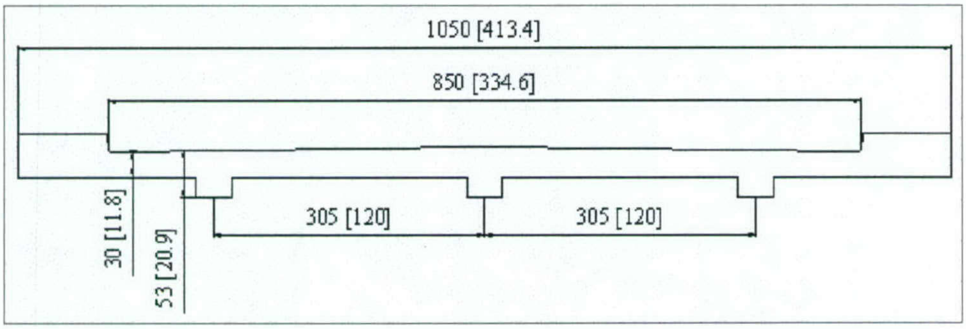


Figure 12. The deck of the Birecik Bridge

According to ACI 318 (2002), R9.3.1, the purposes of strength reduction factor are:

a. To consider the probability of understrength members due to variations in material strengths and dimensions.

b. To allow for any possible inaccuracies of the design equations.

c. To reflect the degree of ductility and required reliability of the member.

d. To reflect the importance of the member.

ACI 318 (2002), Chapter 20 states that if the required dimensions and material properties are determined through measurements and tests, then strength reduction factors can be as follows:

a.	Flexure, without axial load	1.0
b.	Axial tension (and flexure)	1.0
c.	Axial compression (and flexure) (not spiral)	0.85
d.	Shear	0.9

This research study assumed all the strength reduction factors as 1.0. As the bridges under consideration were constructed about 40 years ago, the concrete would typically have higher compressive strength than the 28-day compression strength, which was neglected in the calculations. Therefore, in the actual case the strength of the concrete is higher than the value used in the calculations. The dimensions of this bridge were obtained from site investigations and were actual in-place dimensions.

Slab

The bridge slab was checked by the ASD approach prescribed in AASHTO (1996). The minimum allowable main reinforcing steel was assumed present. Distribution steel and temperature steel were not checked as no information was available in this regard.

e. Effective Span. According to AASHTO (1996), Section 3.24.1.2a, effective span, S, is the clear span between T-Girders.

$$S = 104.33 \text{ in.} = 8.69 \text{ ft } (2.65 \text{ m})$$

f. Dead Load. Dead load of the slab was calculated in terms of uniform pressure (ksf).

Self weight of slab, w_1: $w_1 = t \cdot \gamma$

where t: thickness of slab in feet

γ: Unit weight of concrete

$$w_1 = \frac{11.8 \text{ in.}}{12 \text{ in./ft}} \cdot 0.150 \text{ kcf} = 0.1475 \text{ ksf } (7.0623 \text{ kPa})$$

Wearing surface, assumed, w_2: $w_2 = 0.019$ ksf (0.91 kPa)

Total dead load, $w = w_1 + w_2 = 0.1475 + 0.019 = 0.1665 \text{ksf}$ (7.972 kPa)

Dead load moment, M_D:

$$M_D = \pm \frac{w \cdot S^2}{10} = \frac{0.1665\ ksf \cdot (8.69\ ft)^2}{10} = 1.257\ k-ft \text{ (1.704 kN-m)}$$

g. Live Load. Live load moment, M_L, at midspan of the slab was calculated according to AASHTO (1996), Section 3.24.3.1 using the rear wheel of the HET as the loading.

Rear wheel load of HET, P_{HET} = 15.7 kips (69.8 kN)

$$M_L = 0.8 \cdot \frac{S+2}{32} \cdot P_{HET} = 0.8 \cdot \frac{8.69\,ft + 2}{32} \cdot 15.7\ kips = 4.196 \text{ k-ft (5.689 kN-m)}$$

Impact Factor, I: (AASHTO (1996), Section 3.8.2.1)

$$I = \frac{50}{S+125} = \frac{50}{8.69\,ft + 125} = 0.37 > 0.3 \quad \text{(Maximum allowable)}$$

Hence I = 0.3

$M_{L+I} = (1+I)\cdot M_L = (1+0.3)\cdot 4.196 = 5.454$ k-ft (7.395 kN-m)

Total design moment, $M_T = M_D + M_{L+I}$

$M_T = 1.257$ k-ft + 5.454 k-ft = 6.71 k-ft (9.1 kN-m)

h. Main Steel. Yield stress for ST37 steel is given in Turkish Standards-648 (TSI 1980).

$f_y = 34{,}000$ psi (234.4 MPa)

Allowable stress for ST37 steel is given in Turkish Standards-648 (TSI 1980).

$f_s = 20{,}000$ psi (137.9 MPa)

Allowable stress for concrete is 0.4 times of the concrete nominal strength (f_c').

$f_c = 0.4\cdot f_c' = 0.4\cdot 3{,}250 \text{ psi} = 1{,}300 \text{ psi}$ (8.96 MPa)

Modular ratio (n), the ratio of the modulus of elasticity of steel to that of concrete, is given in AASHTO (1996), Section 10.38.1.3. Modular ratio is dependent on the concrete nominal strength (f_c').

$n = 9$ (for $f_c' = 3{,}250$ psi (22.4MPa))

The design coefficients k (neutral axis factor), and j (lever-arm factor) were calculated.

$$k = \frac{n}{n+\dfrac{f_s}{f_c}} = \frac{9}{9+\dfrac{20{,}000\ psi}{1{,}300\ psi}} = 0.369$$

$$j = 1-\frac{k}{3} = 0.877$$

The distance from the concrete surface to the center of the rebars was assumed as 1.8 in. Therefore, the depth of the slab (d) was calculated as,

$d = t - 1.8 = 11.8 \text{ in.} - 1.8 \text{ in.} = 10.0 \text{ in.}$ (0.254 m)

Required slab depth, d_r:

$$d_r = \sqrt{\frac{M_T}{0.5\cdot f_c\cdot k\cdot j}} = \sqrt{\frac{6{,}710 \text{ lb-ft}}{0.5\cdot 1{,}300 \text{ psi}\cdot 0.369\cdot 0.877}}$$

$d_r = 5.65$ in. (0.14 m) < 10 in. (0.254 m) OK.

Minimum slab depth, d_{min}: (AASHTO (1996), Table 8.9.2)

$$d_{min} = \frac{S+10}{30} = \frac{8.69\,ft+10}{30} \cdot 12\,in./ft = 7.48\,in.\,(0.19\,m) < 10\,in.\,(0.254\,m)\ \text{OK.}$$

The minimum steel amount is calculated according to ACI 318 (2002).

$$\text{Minimum steel} = \frac{3\sqrt{f_c'}}{f_y} \cdot b \cdot d > 200\frac{b \cdot d}{f_y}$$

$$\frac{3\sqrt{f_c'}}{f_y} \cdot b \cdot d = \frac{3\sqrt{3{,}250\text{ psi}}}{34{,}000\text{ psi}} \cdot 12\text{ in.} \cdot 10\text{ in.} = 0.60\text{ in.}^2 \text{ per foot of slab}$$

$$200\frac{b \cdot d}{f_y} = 200\frac{12\text{ in.} \cdot 10\text{ in.}}{34{,}000\text{ psi}} = 0.70\text{ in.}^2 \text{ per foot of slab, governs.}$$

Hence, 0.70 in.^{2} steel per foot of slab (the minimum required by ACI 318 (2002) was assumed to be present at the top and at the bottom of the slab.

Required steel, A_s:

$$A_s = \frac{M_T}{f_s \cdot j \cdot d} = \frac{6{,}710\text{ lb} - \text{ft}}{20{,}000 \cdot 0.877 \cdot 10} \cdot 12\text{ in./ft}$$

$$A_s = 0.46\text{ in.}^2\ (2.97\text{ cm}^2) < 0.70\text{ in.}^2\ (4.52\text{ cm}^2)\ \text{OK.}$$

i. Column Punching. According to AASHTO (1996) Section 8.15.5.6.1, the slab should be checked for punching shear in the vicinity of concentrated loads and reactions. The punching of the column into the slab was neglected, since the columns of the bridge frame into the beams, which in turn support the slab. This is shown in Figure D8 in Appendix D.

j. One-Way Shear Failure Check. The concrete slab was checked against the shear failure in longitudinal direction. The shear capacity of the concrete, V_c, was calculated as;

$V_c = 2\sqrt{f_c'} \cdot b \cdot d = 2\sqrt{3{,}250} \cdot 12 \cdot 10 = 13.7$ kips (AASHTO (1996), Equation 8-49),

where b = 12 in. (unit width of slab)
d = 10 in. (depth of slab)

The maximum shear force in the slab, V, was obtained from SAP2000 model as 3.25 kips.

V = 3.25 kips < 13.7 kips O.K.

k. Tire Punching. If the load applied to the slab by a tire was greater than

the punching shear capacity of the slab, the tire would penetrate ('punch') through the slab. The punching shear of a tire of the HET was checked according to AASHTO (1996) Section 8.15.5.6. According to AASHTO (1996) Section 3.30, the contact area of the tire, A, is:

$$A = 0.01 \cdot P$$

where P = Load on the tire (lb)

The most critical tire was one of the rear tires, for which the load was 7,850 lb (34.92 kN). For this load, the contact area was calculated as 78.5 in^2 (506.5 cm^2), using the equation given above. The contact area is given as a rectangle, for which the ratio of the long side to the short side is 2.5. Therefore the rectangle was 14 in. × 5.6 in. (0.36 m × 0.14 m). Punching stress is calculated by;

$$v = \frac{V}{b_0 \cdot d} \qquad \text{(AASHTO (1996), Equation 8-12)}$$

where, v = Shear stress

b_0 : Perimeter of the critical section (AASHTO (1996) Section 8.15.5.6.1b)

d : Depth of the section

$$v_c = (0.8 + \frac{2}{\beta_c})\sqrt{f_c'} < 1.8\sqrt{f_c'} \qquad \text{(AASHTO (1996), Equation 8-13)}$$

where v_c : Punching shear stress capacity

f_c': Nominal strength of concrete

Therefore, punching shear capacity, V_c, is:

$V_c = v_c \cdot b_0 \cdot d = 72.2$ kips (321.2 kN)

Tire Load = 7.85 kips (34.92 kN) < 72.2 kips (321.2 kN) O.K.

SAP2000 Model

The Birecik Bridge was modeled in SAP2000 described in Chapter 1 as a plane frame. Frame/cable objects of SAP2000 were used for the model. These objects are used to model beams, columns, braces, trusses, and/or cable members.

In the 'Bridge Analysis' section, *SAP2000 Analysis Reference Manual* says that one "should model the bridge structure primarily with Frame elements" (Computer and Structures, Inc. 2002). According to this reference, the displacements, reactions, and frame element internal forces can be determined due to the

influence of vehicle live loads (Computer and Structures, Inc. 2002). Other element types (shell, plane, solid, etc.) may be used; they contribute to the stiffness and may carry part of the load, but they are not analyzed for the effect of vehicle load (Computer and Structures, Inc. 2002).

Vehicle live loads can only be applied to frame elements, thus live loads cannot be represented as acting directly on bridge decks modeled with shell or solid or other elements (Computer and Structures, Inc. 2002). Element internal forces due to vehicle live loads are computed only for frame elements in SAP2000 (Computer and Structures, Inc. 2002). The bridge was modeled in 2-D using frame elements because of these restrictions. This 2-D model resulted in a conservative representation of the actual structure. More accurate representation of the bridge would have been obtained by modeling it in 3-D, which was beyond the scope of this research study.

The arch model is shown in Figure 13. Displacements and internal forces were obtained from the analysis. The bridge is composed of three rows of columns, connecting the slab to the concrete arch. The 2-D frame is composed of slab portion, an arch portion, and one of the three rows of columns connecting the slab to the arch.

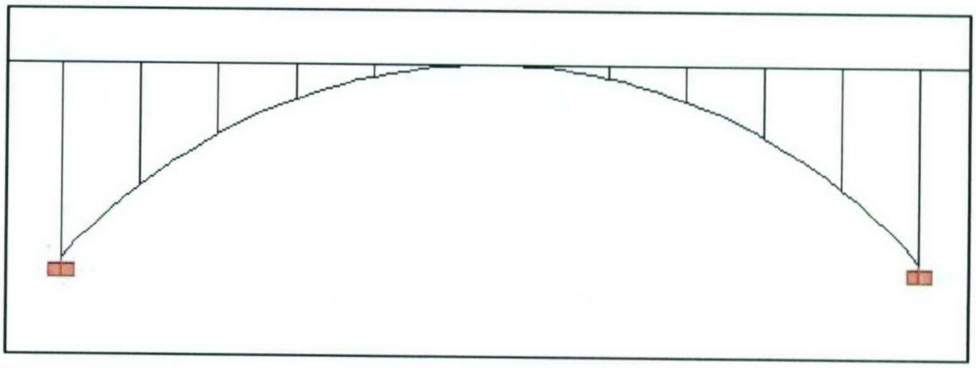


Figure 13. Model arch for SAP2000 analysis

a. Distribution Factor. Only one HET vehicle was assumed to be on the bridge at a time. It was assumed that the HET travels along the center of the bridge. The distribution factor for the live load was determined after running several transverse section analyses (2-D) in SAP2000.

The transverse section was modeled as a continuous beam supported by three columns, one at the each end and one at the middle of the beam (two span, one story frame). The bottoms of the columns were supported to model the effect of the arch (in the actual structure arch supports the columns).

A pair of concentrated loads (6 ft apart from each other to get standard truck geometry in transverse direction) was applied to the center of the beam. Each point load of the pair represented each tire of an axle (two tires per an axle). The height and the stiffness of the columns, the stiffness of the beam, and the support conditions were the basic parameters that affected the distribution of loads to each column. One of these parameters was changed in each analysis and the

forces were calculated in each column to obtain the corresponding distribution factor.

The percentage of live load carried by the central column ranged between 40 to 80 percent depending on the height, stiffness of the columns, and stiffness of the slab and arch.

The distribution factor was assumed to be of the worst case, 0.80, which means that 80 percent of the live load was carried by the main bay. This would be much less if the actual transverse dimensions of the HET were used.

b. Impact Factor. The impact factor was assumed to be 0.1 according to the study done in New Mexico *University* in 2001 (Minor, and Woodward 2001). This impact factor (I = 0.10) was used for the analysis of the 2-D structure in SAP2000.

c. Effective Flange Width. Effective flange width is the minimum of, (AASHTO (1996), Section 8.10.1.1)

$$b_e = \frac{\text{Span Length}}{4} = \frac{196.85\text{ ft}}{4} = 49.2\text{ in. (1.2 m)}$$

b_e = Girder spacing = 104.3 in. (2.6 m)
b_e = 12 times the slab thickness + the web width = 157.4 in. (4 m)
Hence b_e = 49.2 in. (1.2 m)

d. Convergence Check of the Model. Each arch and the corresponding deck slab were modeled four times. Each time, the number of elements was increased to monitor the convergence of the analysis results. The structure was modeled with 11, 55, 110, and 220 elements in the first, second, third, and the fourth model, respectively. Four points were selected to monitor the convergence of the computer model. These points are the top of the columns numbered and shown in Figure 14.

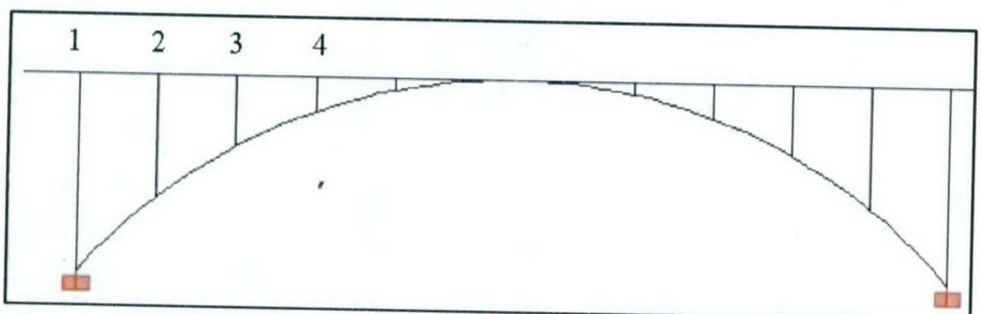


Figure 14. The columns of the main arch monitored for the convergence in SAP2000

Table 16 shows the displacements of these points obtained from the analysis of the corresponding model. As the model with 55 members gave reasonable results and an acceptable convergence, shown in Figure 15, the structure was modeled with 55 members.

Table 16 Displacements of Selected Points (in.) in the Corresponding Computer Model								
Number of elements used to define the arch	Point1		Point2		Point3		Point4	
	In.	Mm	In.	mm	In.	mm	In.	mm
11	0.8649	21.97	0.8067	20.49	0.5753	14.61	0.2993	7.602
55	0.8834	22.43	0.8333	21.16	0.5951	15.11	0.3036	7.711
110	0.8835	22.44	0.8334	21.17	0.5952	15.12	0.3036	7.711
220	0.8836	22.44	0.8335	21.17	0.5952	15.12	0.3036	7.711

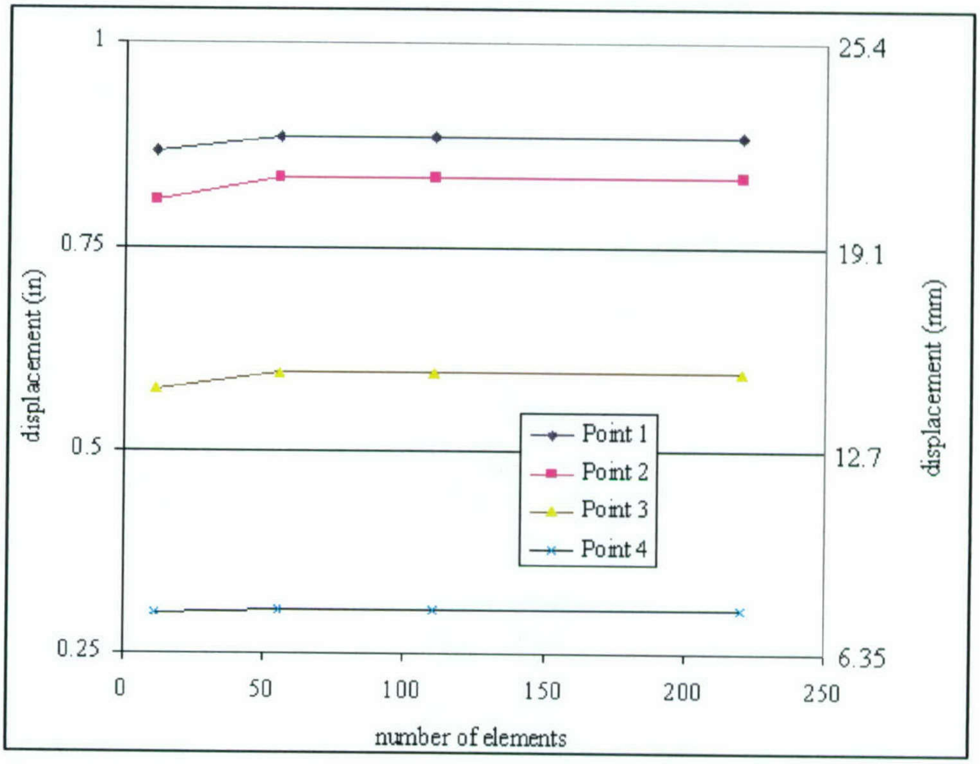


Figure 15. Convergence curves for the values of vertical displacement of selected points

e. Section Modeling. Five different sections were defined to model T-Girder slab, column, arch, column wall, and the crown of the arch. The effective width of the arch was 85 in. (2.2 m). The slab and the girder were modeled as a T-Girder using the effective flange width of 49.2 in. (1.2 m) according to AASHTO (1996), Section 8.10.1.1. The columns were modeled according to their clear lengths. A different section was introduced at the crown of the arch in order to model the arch and the T-Girder together. These sections are shown in Figure 16.

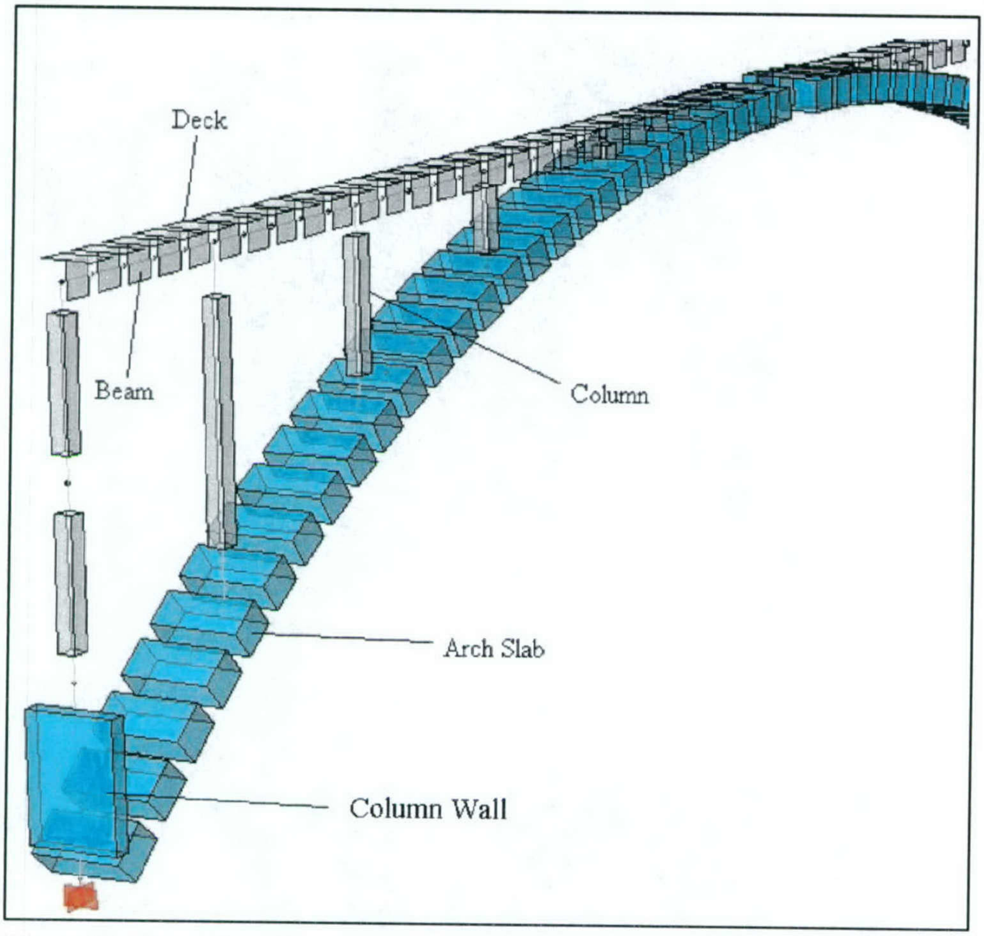


Figure 16. Element section composition of the structure modeled in SAP2000

The bridge was modeled and analyzed with three arches due to the symmetry. The 3-Arch bridge model is shown in Figure 17. A picture of the actual bridge, which has five arches, is given in Figure 18.

f. Loading. Only primary loads; dead load, live load and impact, were considered in to the bridge model. Dead load was calculated according to the unit weights of materials given in AASHTO (1996).

The live load considered in the model was one HET vehicle, shown previously in Figure 2. Eighty (80) percent of the live load was used as the distribution factor on the middle bay of the bridge. The loads were used as service loads and not factored by any load factors.

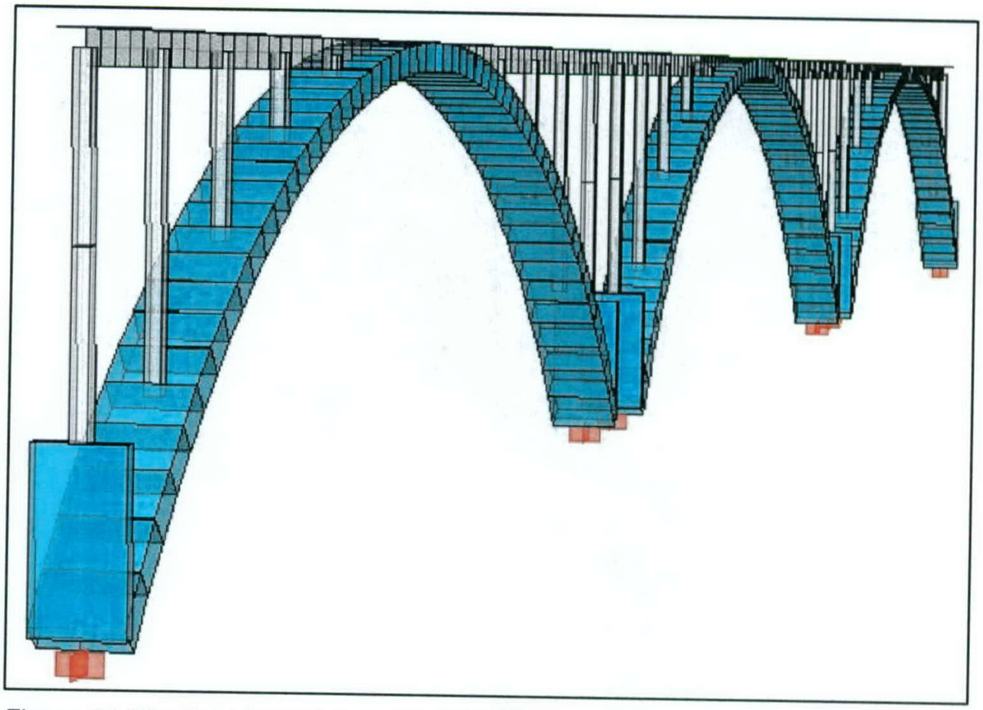


Figure 17. The 3-Arch model used in SAP2000 analysis

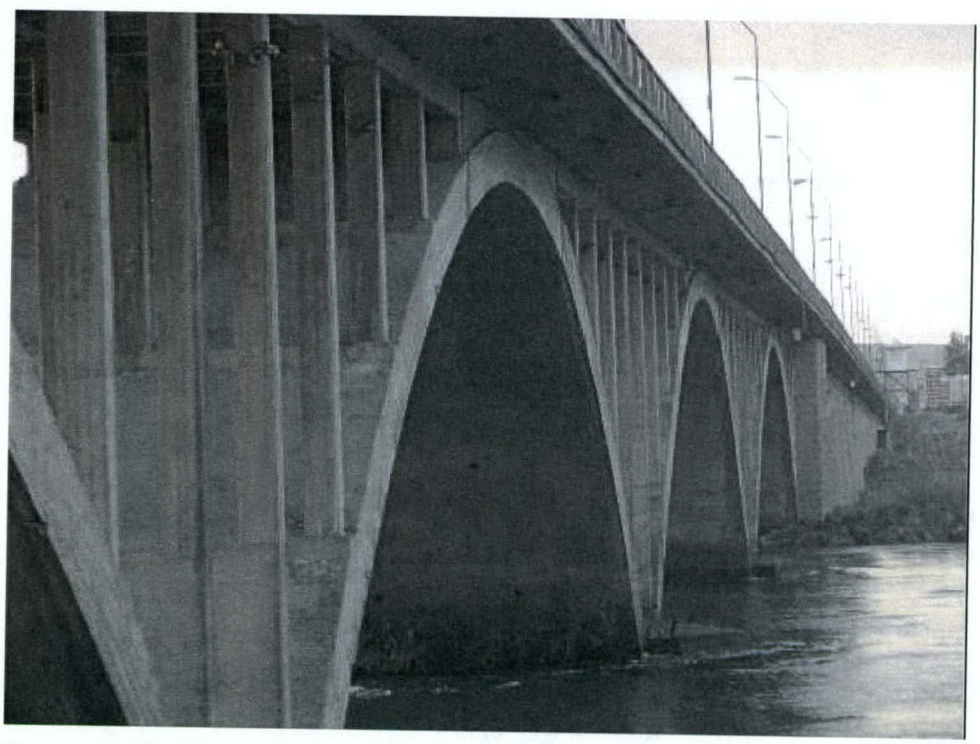


Figure 18. The Birecik Bridge

Impact factor was calculated in ASSHTO (1996) Section 4.2.2.2 as 0.17. The impact and distribution factors were assumed to be same for the entire 2-D model. Therefore, the total load was,

$$TL = DL + DF \cdot LL \cdot (1 + I)$$

where
TL: Total load
DL: Dead load
LL: Live load
DF: Distribution factor
I: Impact

This loading combination was used in the SAP2000 model.

g. *Results.* The deformed shape (not to scale) of the main arch is shown in Figure 19. The maximum deflection at the crown of the arch was calculated to be less than 1 in. The moment, shear, and axial force envelopes obtained from the analysis are shown in Appendix C.

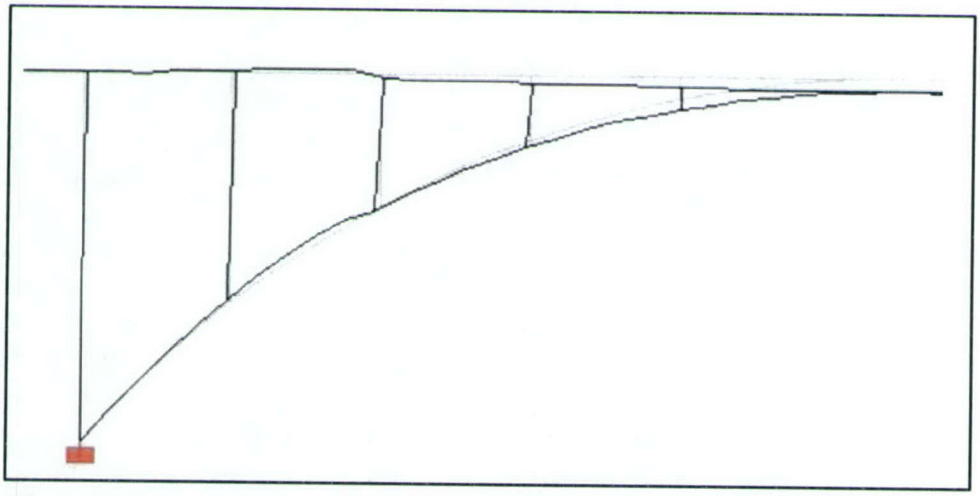


Figure 19. Deformed shape of the main arch (half-arch), SAP2000

The critical internal force combinations for each section were obtained from the analysis and tabulated in Table 17. In this table, the entries do not necessarily represent the maximum values, but the most critical force combination for the corresponding member type. The most critical force combination for a member was the force pair (moment/axial force) for which the position of the load combination on the interaction diagram was the closest to the boundary defined by the interaction diagram. Interaction diagram shows the boundary of 'safe region' for a section regarding the moment/axial force applied to the section. Basically if the point of a force pair (moment/axial force) is within the boundaries, then the section is said to be satisfactory. The interaction diagrams for the sections of Birecik Bridge are given in Appendix C.

Table 17
The Critical Internal Forces for Each Member Type

	Critical Moment		Critical Axial Force	
	(kN-m)	(kips-ft)	(kN)	(kips)
Arch	1,705.2	1,257.7	2,259.9	508.0
Arch Crown	1,391.8	1,026.5	1,573.5	353.7
T-Girder (Slab)	392.7	289.6	547.2	123.0
Column	164.5	121.3	67.6	15.2
Column Wall	34.6	25.5	92.1	20.7

The maximum axial force and the maximum shear force for each member type are tabulated and given in Table 18. In this table the values for axial and shear forces are not necessarily for the same member, but the maximum values for each member type.

Table 18
The Maximum Axial and Shear Force for Each Member Type

	Maximum Axial Force		Maximum Shear Force	
	(kN)	(kips)	(kN)	(kips)
Arch	3,652.3	821.0	498.2	112.0
Arch Crown	2,692.3	605.2	238.9	53.7
T-Girder (Slab)	706.9	158.9	252.7	56.8
Column	494.7	111.2	149.9	33.7
Column Wall	555.6	124.9	4.8	1.1

h. Critical Moment/Axial Force Combinations Check. Uniaxial interaction diagrams of each of the five section types were obtained using PCACOL program, described in Chapter 1. While running the program, the slenderness effect was not considered for the columns (only short columns were analyzed in this section). The check for the column stability considering slenderness effect (for the slender column of the arch) is given in Section 4.2.2.9. The most critical loading combinations, given in Table 17, were used in the analysis. The minimum steel (ACI 318 2002) was assumed.

The minimum steel for columns, A_s;

$$A_s = 0.01 \cdot A_g \qquad \text{(ACI (2002), 10.9.1)}$$

where A_g: Gross area of section

The minimum steel for T-Girder beams, A_s;

$$A_s = \frac{3\sqrt{f_c'}}{f_y} \cdot b_w \cdot d \geq 200\frac{b_w \cdot d}{f_y}, \text{ for web reinforcement} \quad \text{(ACI 318 (2002), 10-3)}$$

$$A_s = \frac{6\sqrt{f_c'}}{f_y} \cdot b_w \cdot d \text{, for flange reinforcement} \qquad \text{(ACI 318 (2002), 10-4)}$$

where b_w: Web width

d: Depth of girder

f_c': Nominal strength of concrete

Interaction diagrams given in Appendix C showed that Arch, Arch Crown, T-Girder, and Column Wall sections were satisfactory (the point of the critical load pair, marked with a plus on the diagram, was within the boundary of the satisfactory zone) under the HET loading provided that at least the minimum steel according to ACI 318 (2002) was supplied. This showed that the T-Girder was satisfactory in flexure.

The column (short column), which is 15.75 in. by 15.75 in. (40.0 cm by 40.0 cm), was not satisfactory with minimum steel according to ACI 318 (2002) (1 percent). To get similar information about the steel ratio used in the columns, the blueprint of the columns of a similar bridge were obtained and studied (GDH 1953a). The name of the bridge is Dokuzdolambac II. It is a reinforced concrete spandrel arch bridge, same as the Birecik Bridge, constructed in 1953. The steel ratio in the columns varies between 2.64 and 3.30 percent (GDH 1953a). To be on the safe side the lower value of 2.64 percent steel ratio was assumed to be present in the columns of Birecik Bridge. This was a valid assumption as both bridges were constructed in 1950s on similar 2-lane highways in Turkey, and the bridge types, materials, span lengths, and design live loads were similar.

The short column of the Birecik Bridge was not satisfactory after the second assumption (steel ratio of 2.64 percent). The stress in the steel was calculated as 40.5 ksi (279.2 MPa). The yield limit of the steel is 34 ksi (234.4 MPa). It was found that it would be satisfactory with a steel ratio of at least 2.94 percent, which is within the range of steel reinforcing in the Dokuzdolambac II Bridge. It was concluded that the (short) column was not safe for the HET loading. The interaction diagrams for 1 and 2.64 percent steel ratios are given in Appendix C.

i. Column Stability Considering Slenderness Effect. Column 2 (shown in Figure 14), the most critical slender column of the Birecik Bridge, was checked considering the slenderness effect using moment magnification method.

Clear length of the column, $l = 305.7$ in. (7.8 m)

Dimensions of the column ($h \times h$) = 15.75 in. × 15.75 in. (40.0 cm × 40.0 cm)

Radius of gyration, $r = 0.3h = 0.3 \cdot 15.75 \text{ in.} = 4.73$ in. (0.12 m)

Axial force in the column, $P = 111.2$ kips (494.7 kN) (from Table 17)

Moment at the top, M_1 = 28.4 kips-ft (38.5 kN-m)

Moment at the bottom, M_2 = 52.5 kips-ft (71.1 kN-m)

These were the highest end moments causing single curvature. There was a possibility that this column be in double curvature with smaller end moments. Single curvature with higher end moments was checked as it was the most critical case.

For a sway (unbraced) frame;

According to AASHTO (1996), Section 8.16.5.2.6, moment magnification method can be used only if the slenderness of the column is smaller than 100 ($\frac{k \cdot l}{r} < 100$). The column was analyzed assuming the worst case ($\frac{k \cdot l}{r} = 100$). If the slenderness was greater than 100, then the column would be subject to an analysis considering material non-linearity and cracking, effects of member curvature and lateral drift, duration of the loads, shrinkage and creep, and interaction with the supporting foundation (ACI 318 (2002), 10.10.1). This analysis, which requires more information about the structure, was beyond the scope of this research study. Therefore the length factor, k, was calculated by assuming that the slenderness was equal to 100.

Length factor, k = 1.54

Magnified moment, M_c, is defined by:

$$M_c = \delta \cdot M_2 \qquad \text{(AASHTO (1996), Equation 8-40),}$$

where δ, the magnification factor, is defined by:

$$\delta = \frac{1}{1 - \frac{\sum P}{\phi \cdot \sum P_c}} \geq 1.0 \qquad \text{(AASHTO (1996), Equation 8-41A),}$$

where P_c, the critical load, is defined by:

$$P_c = \frac{\pi^2 \cdot EI}{(k \cdot l)^2} \qquad \text{(AASHTO (1996), Equation 8-42),}$$

where EI, flexural stiffness, is defined by:

$$EI = \frac{0.4 \cdot E_c \cdot I_g}{1 + \beta_d} \qquad \text{(AASHTO (1996), Equation 8-43),}$$

where
P =Axial load
E_c=Modulus of elasticity of concrete

I_g =Moment of inertia of gross area
β_d =Ratio of maximum dead load moment to total load moment.

β_d, obtained from SAP2000 analysis, was 0.16.

These calculations showed that, $\delta = 1.8$

Therefore,

$M_c = 1.8 \cdot 52.5 = 94.5$ kips-ft (128.1 kN-m)The slender column was found to be satisfactory for these forces as shown in the interaction diagram, given in Appendix C (Figure C6).

j. Punching Shear in Arch. If the shear force applied to the arch slab by a column was greater than the punching shear capacity of the arch slab, then the column would penetrate ('punch') through the arch slab. The punching shear was checked according to ACI 318 (2002).

According to ACI 318 (2002), punching shear capacity of an arch is the smallest of:

- $V_c = (2 + \frac{4}{\beta_c})\sqrt{f_c'} \cdot b_0 \cdot d$ (ACI 318 (2002), 11-35)
- $V_c = (2 + \frac{\alpha_s \cdot d}{b_0})\sqrt{f_c'} \cdot b_0 \cdot d$ (ACI 318 (2002), 11-36)
- $V_c = 4\sqrt{f_c'} \cdot b_0 \cdot d$ (ACI 318 (2002), 11-37)

where

V_c = Punching capacity
β_c = The ratio of long side to short side of the column
f_c' = Nominal strength of concrete
b_0 = Perimeter of the critical section (ACI 318 (2002), 11.12.1.2)
d = Depth of the section
α_s = Constant(40 for interior columns, 30 for edge columns)

The maximum shear force = 112 kips (498.2 kN) (from Table 17)

For edge column, the capacity was calculated according to ACI 318 (2002), 11-37 (critical), given above. The capacity, Vc, was:

V_c = 664 kips (2,953.6 kN) > 112 kips (498.2 kN) O.K.

For interior column, the capacity was calculated according to ACI 2002, 318-02:11-35 (critical), given above. The capacity, V_c, was:

V_c = 996 kips (4,430.4 kN) > 112 kips (498.2 kN) O.K.

k. Shear Check in the Girder. Allowable shear stress in concrete is:

$v_c = 0.95\sqrt{f_c'}$ (AASHTO (1996), Section 8.15.5.2.1)

Allowable shear to be carried by shear reinforcement, v_s is:

$v_s = 4\sqrt{f_c'}$ (AASHTO (1996), Section 8.15.5.3.9)

Maximum allowable shear for the section, v is:

$$v = v_c + v_s = 0.95\sqrt{f_c'} + 4\sqrt{f_c'} = 4.95\sqrt{f_c'} = 282 \text{ psi} \ \ (1.9 \text{ MPa})$$

$$\text{Required web area} = \frac{V}{v} = \frac{56.80 \text{ kips}}{0.282 \text{ ksi}} = 201 \text{ in.}^2 \ \ (0.13 \text{ m}^2)$$

Where V: Maximum shear force in the girder (from Table 18)

$$\text{Actual web area} = b_w \cdot d = 15.75 \text{ in.} \cdot 30.2 \text{ in.}$$

$$= 475 \text{ in.}^2 \ (0.31 \text{ m}^2) > 201 \text{ in.}^2 \ (0.13 \text{ m}^2)$$ O.K.

where b_w: Web width

d: Depth of the girder

Hence concrete web area is satisfactory in regard to the shear stress.

Stress in the shear reinforcement (stirrups) was checked:

Allowable shear stress in concrete, $v_c = 0.95\sqrt{f_c'}$
(AASHTO (1996), Section 8.15.5.2.1)

v_c = 54.2 psi (0.37 MPa)

Shear stress carried by steel, v_s:

$$v_s = \frac{V}{b_w \cdot d} - v_c = \frac{56{,}800 \text{ lb}}{15.75 \text{ in.} \cdot 30.2 \text{ in.}} . - 54.2 \text{ psi} = 65.2 \text{ psi} \ \ (0.45 \text{ MPa})$$

Assuming that minimum shear reinforcement was used,

$$\frac{A_v}{s} = 50\frac{b_w}{f_y}$$

(Minimum shear reinforcement according to ACI 318 (2002), 11-13)

where
A_v: Area of shear reinforcement
s: Spacing of the stirrups

$$\frac{A_v}{s} = 0.0232 \text{ in.}^2\text{/in., supplied (the minimum shear reinforcement)}$$

Check stress in the stirrup with the minimum steel, f_s:

$$f_s = \frac{v_s \cdot b_w}{A_v/s} = \frac{65.2\,psi \cdot 15.75\,in.}{0.0232\,in.^2/in.}$$

$$f_s = 44,263 \text{ psi } (305.2 \text{ MPa}) > 34,000 \text{ psi } (234.4 \text{ MPa}) \qquad \text{(yield limit stress)}$$

Minimum steel was not satisfactory for shear reinforcement in the girders. As the minimum steel was not satisfactory the information for shear reinforcement was obtained from the blueprints of Dokuzdolambac II Bridge. It was found that the diameter of the stirrups was 1.2 cm and they were installed with a 25 cm spacing (GDH 1953a). This amount of steel was almost the same as No. 4 stirrups placed every 10 in. (0.254 m). Therefore it was assumed that No. 4 stirrups were present at every 10 in. (0.254 m) throughout the beam.

Checking the stress in the stirrup with the new steel ratio assumption;

$$f_s = \frac{v_s \cdot b_w}{A_v/s} = \frac{65.2 \text{ psi} \cdot 15.75 \text{ in.}}{0.4 \text{ in.}^2/10 \text{ in.}}$$

$$f_s = 25,700 \text{ psi } (177 \text{ MPa}) > 20,000 \text{ psi } (137.9 \text{ MPa}) \qquad \text{(allowable stress)}$$

$$< 34,000 \text{ psi } (234.4 \text{ MPa}) \qquad \text{(yield limit)}$$

Stress in the stirrup was 30 percent higher than the allowable stress. It was still lower than the minimum yield limit, which was acceptable (as the steel did not yield) considering the conservative approach used in the analysis (HET was assumed to have 18 tires instead of 28, which resulted in higher concentrated loads and higher distribution factor than the actual case).

Summary of Results

An open spandrel reinforced concrete arch bridge, constructed in 1956, was analyzed according to AASHTO (1996) with no load factors using the HET as the live load. The calculations did not consider any factors of safety. Neither the loads nor the section capacities were factored. The only safety factors included in the calculations were due to the use of allowable stresses instead of the nominal strength for the materials.

Slab

It was assumed that the minimum reinforcement steel according to ACI 318 (2002) was used. Thickness of the slab and the main steel were satisfactory. The section was adequate against shear failure.

2-D Computer Model

The analysis of a 2-D model in SAP2000 and in PCACOL showed that all bridge members except the columns were satisfactory assuming minimum amount of steel in each section. The columns, on the other hand, were not satisfactory assuming the minimum steel was used. In order to make another assumption, the blueprints of an identical bridge were studied. The steel ratio used in the columns of this bridge (Dokuzdolambac II Bridge) was 2.64 percent (GDH 1953a). The same steel ratio was assumed to have been used in the design of Birecik Bridge. The short column of the Birecik Bridge was not satisfactory after the second assumption (steel ratio of 2.64 percent). It was found that it would be satisfactory with a steel ratio of at least 2.94 percent, which is within the range of steel reinforcing in the Dokuzdolambac II Bridge. It was concluded that the (short) column was not safe for the HET loading. The interaction diagrams for 1 and 2.64 percent steel ratios in the column are given in Appendix C.

The shear strength of the T-Girders was lower than the shear stress in the section, assuming the minimum amount of stirrup according to ACI 318 (2002) was used. Then another assumption was made according to the shear reinforcement used in Dokuzdolambac II Bridge (GDH 1953a). It was assumed that No. 4 bars were used as stirrups at a 10 in. (0.254 m) spacing. With this assumption, the stress in the stirrups was 8,300 psi (57 MPa) lower than the minimum yield limit. This was an acceptable stress considering the conservative approach used in the analysis. The punching shear capacity of the arch was found to be satisfactory.

The overall assessment for the Birecik Bridge (with the information in hand and the assumptions made in the analysis, given in this Chapter) indicates that the columns of the bridge were critical and may not be adequate to support a HET vehicle.

5 Analysis of Composite I-Girder Bridge

Introduction

The blueprints of this I-Girder composite continuous bridge were prepared by the GDH before 1950s. These drawings were not for a specific bridge, but were standard drawings, which were applied as needed. Blueprints of the typical superstructure were obtained from the GDH Ankara Headquarter. Design live load was HS15, which is 10 percent heavier than HS15 of AASHTO (1996) as described in Chapter 3 (GDH 1953b).

Curb-to-curb width of this bridge is 6.00 m (19.69 ft), which makes it a 2-lane bridge, as shown in Figure 20. The maximum span is 12.00 m (39.37 ft), as shown in Figure 21. The total length of the bridge could be up to 152.0 m (498.7 ft) with multiple spans, the longest of which is 12.00 m (39.37 ft) (GDH 1953b).

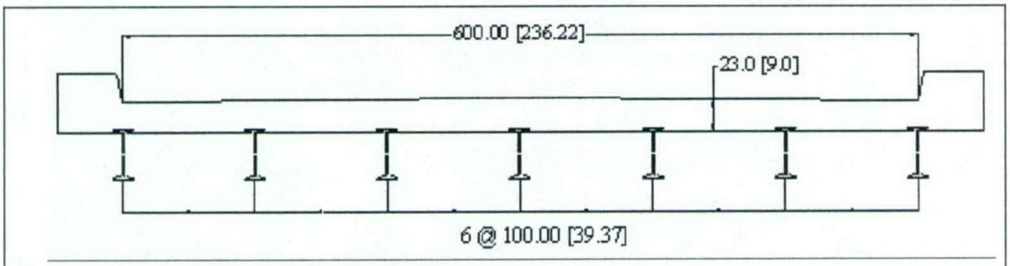


Figure 20. Cross-section of the bridge (GDH 1953b)

A 9-in. thick reinforced concrete slab rests on seven I-Girders, spaced at 1.00 m (3.28 ft). The cross-section of the bridge is shown in Figure 20. The steel girder has a depth of 38 cm (14.96 in.). This is a standard I-Girder (I-38). The steel used for reinforcement and I-Girders is ST37, which was described in Chapter 3. The compression strength of concrete, f_c', was given as 22.1 MPa (3,200 psi) (GDH 1953b).

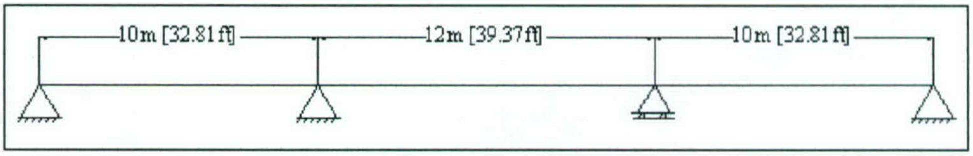


Figure 21. Layout of the bridge (GDH 1953b)

Analysis of the I-Girder Bridge

Slab, exterior girder and interior girder, were checked using the ASD approach given in AASHTO (1996). The substructure of the bridge was not analyzed as the focus of the research was primarily on the superstructure.

The bridge was also modeled using SAP2000 and the internal forces were obtained from a 2-D analysis. Only the primary loads; dead load, live load and impact, were applied to the structure. Only one HET vehicle at a time was assumed to be on the bridge, as prescribed in AASHTO (1996). Actual strengths of the sections were calculated without introducing strength reduction factors, described in Chapter 4. Loads were not modified by any factor.

Slab

a. Effective Span. Computation of the effective span length, S was made according to AASHTO (1996), Section 3.24.1.2b.

$$S = 3.28\text{ ft} - 0.5 \cdot \frac{5.9\text{ in.}}{12\text{ in./ft}} = 3.034\text{ ft (0.92 m)}$$

b. Dead Load. Dead load of the slab was calculated in terms of pressure (ksf).

Self weight of slab, w_1: $w_1 = t \cdot \gamma$

Where t: thickness of slab in feet

γ: Unit weight of concrete

$$w_1 = \frac{9\text{ in.}}{12\text{ in./ft}} \cdot 0.150\text{ kcf} = 0.1125\text{ ksf (5.3865 kPa)}$$

Wearing surface, assumed, w_2: $w_2 = 0.0275$ ksf (1.3167 kPa)

Total dead load, w: $w = w_1 + w_2 = 0.1125 + 0.0275 = 0.140$ ksf (6.703 kPa)

Dead load moment, M_D:

$$M_D = \pm\frac{w \cdot S^2}{10} = \frac{0.14\ ksf \cdot (3.034\ ft)^2}{10} = 0.129\ k - ft\ \text{(0.175 kN-m)}$$

c. Live Load. Live load moment at midspan, ML, was calculated according to the formula given in AASHTO (1996), Section 3.24.3.1 using the rear wheel of HET as the loading.

Rear wheel load of HET, P_{HET} = 15.7 kips (69.8 kN)

$$M_L = 0.8 \cdot \frac{S+2}{32} \cdot P_{HET} = 0.8 \cdot \frac{3.034\,ft+2}{32} \cdot 15.7\,kips = 1.976\ \text{k-ft (2.679 kN-m)}$$

where S: Effective span length in feet.

Impact Factor, I: (AASHTO (1996), Section 3.8.2.1)

$$\mathrm{I} = \frac{50}{\mathrm{S}+125} = \frac{50}{3.034\,\mathrm{ft}+125} = 0.39 > 0.3 \qquad \text{(Maximum allowable)}$$

Hence I = 0.3

$M_{L+I} = (1+\mathrm{I}) \cdot \mathrm{M_L} = (1+0.3) \cdot 1.976\ \mathrm{k-ft} = 2.569$ k-ft (3.483 kN-m)

The total design moment, $M_T = M_D + M_{L+I}$

M_T = 0.129 k-ft + 2.569 k-ft = 2.698 k-ft (3.658 kN-m)

d. Main Steel. Yield stress for ST37 steel is given in Turkish Standards, TS-648 (TSI 1980).

f_y: 34,000 psi (234.4 MPa)

Allowable stress for ST37 steel is given in Turkish Standards, TS-648(TSI 1980).

f_s: 20,000 psi (137.9 MPa)

According to ACI 318 (2002), the allowable stress for reinforced concrete is 0.4 times of the concrete nominal strength (f_c').

$f_c = 0.4 \cdot f_c' = 0.4 \cdot 3{,}200\ \text{psi} = 1{,}280\ \text{psi (8.8 MPa)}$

Modular ratio (n), the ratio of the modulus of elasticity of steel to that of concrete, is given in AASHTO (1996), Section 10.38.1.3. Modular ratio is dependent on the concrete nominal strength (f_c').

n = 9 for f_c'=3,200 psi (22.1 MPa)

The design coefficients k (neutral axis factor), and j (lever-arm factor) were calculated.

$$k = \frac{n}{n + \frac{f_s}{f_c}} = \frac{9}{9 + \frac{20,000\,psi}{1,280\,psi}} = 0.365$$

$$j = 1 - \frac{k}{3} = 0.878$$

Required slab depth, d_r:

$$d_r = \sqrt{\frac{M_T}{0.5 \cdot f_c \cdot k \cdot j}}$$

$$d_r = \sqrt{\frac{2{,}698 \text{ lb} - \text{ft}}{0.5 \cdot 1{,}280 \text{ psi} \cdot 0.365 \cdot 0.878}} = 3.63 \text{ in. } (9.22 \text{ cm}) < 7 \text{ in. } (17.78 \text{ cm}) \qquad \text{OK.}$$

Supplied main steel = 0.57 in.^{2} (3.68 cm^2) per foot of slab (GDH 1953b).

Required steel, A_s:

$$A_s = \frac{M_T}{f_s \cdot j \cdot d} = \frac{2{,}698 \text{ lb} - \text{ft}}{20{,}000 \text{ psi} \cdot 0.878 \cdot 7 \text{ in.}} \cdot 12 \text{ in./ft}$$

$$A_s = 0.26 \text{ in.}^2 \ (1.68 \text{ cm}^2) < 0.57 \text{ in.}^2 \ (3.68 \text{ cm}^2) \qquad \text{OK.}$$

e. *Distribution Steel.* (AASHTO (1996), Section 3.24.10.2)

Supplied distribution steel = 0.244 in.^{2} (1.57 cm^2) per foot of slab (GDH 1953b).

$$\text{Percentage} = \frac{220}{\sqrt{S}} = \frac{220}{\sqrt{3.034}} = 125\ \% > 67\ \% \qquad \text{(Maximum allowable)}$$

The amount of distribution steel can be at most 67 percent of the supplied main steel. Therefore, the required distribution steel is,

$$\text{Required steel} = 0.67 \cdot 0.263\, in.^2 = 0.176 \text{ in.}^2 \ (1.14 \text{ cm}^2)$$

$$< 0.244 \text{ in.}^2 \ (1.57 \text{ cm}^2) \qquad \text{OK.}$$

f. Temperature Steel. According to AASHTO (1996) Section 8.20, the required temperature steel per foot of slab should be constant and equal to 0.125 in.^{2} (0.806 cm^2).

Supplied temperature steel = 0.487 in.^{2} (3.142 cm^2) per foot of slab (GDH 1953b).

Required steel = 0.125 in.^{2} (0.806 cm^2) < 0.487 in.^{2} (3.142 cm^2) OK.

g. Column Punching. According to AASHTO (1996), Section 8.15.5.6.1, the slab should be checked for punching shear in the vicinity of *concentrated loads* and reactions. The punching of the column into the slab was neglected, since the columns of the bridge support the I-Girders, which in turn support the slab.

h. Tire Punching. If the load applied to the slab by a tire was greater than the punching shear capacity of the slab, the tire would penetrate ('punch') through the slab. The punching shear of a tire of the HET was checked according to AASHTO (1996), Section 8.15.5.6. According to AASHTO (1996), Section 3.30, the contact area of the tire is:

$$A = 0.01 \cdot P$$

where A = Contact area (in.^{2})
P = Load on the tire (lb)

The most critical tire was one of the rear tires, for which the load was 7,850 lb (34.92 kN). For this load, the contact area was calculated as 78.5 in.^{2} (506.5 cm^2), using the equation given above. The contact area is given as a rectangle, for which the ratio of the long side to the short side is 2.5. Therefore the rectangle was 14 in. × 5.6 in. (0.36 m × 0.14 m). Punching stress is calculated by;

$$v = \frac{V}{b_0 \cdot d} \qquad \text{(AASHTO (1996), Equation 8-12)}$$

where, v = Shear stress

b_0 : Perimeter of the critical section (AASHTO (1996), Section 8.15.5.6.1b)

d : Depth of the section

$$v_c = (0.8 + \frac{2}{\beta_c})\sqrt{f_c'} < 1.8\sqrt{f_c'} \qquad \text{(AASHTO (1996), Equation 8-13)}$$

where v_c : Punching shear stress capacity

f_c': Nominal strength of concrete

Therefore, punching shear capacity, V_c, is:

$V_c = v_c \cdot b_0 \cdot d = 42.9$ kips (190.8 kN)

Tire Load = 7.85 kips (34.92 kN) < 42.9 kips (190.8 kN) O.K.

Girders

Curb and railing load was assumed to be distributed equally to all stringers as per AASHTO (1996), Section 3.23.2.3.1.1.

a. *Interior Girder.*

(1) Dead Load. Weight of the slab was previously calculated. The spacing of the girders was equal to 3.28 ft (1.0 m). Therefore, the weight of the slab over one girder was calculated as,

Weight of the deck, $w_1 = 0.14\,\text{ksf} \cdot 3.28\,\text{ft} = 0.459\,\text{k/ft}\ (6.697\,\text{kN/m})$

Curb and the railing load were distributed to 7 girders.

Curb and railing load, $w_2 = \dfrac{0.35}{7} = 0.05\,\text{k/ft}\ (0.73\,\text{kN/m})$

The dead load of the girder was increased by 50 percent to account for the extra steel (connections, stiffeners, cover plates etc.).

Weight of the girder (increased by 50 percent), $w_3 = 0.084$ k/ft (1.23 kN/m)

Total load on the girder, $w = w_1 + w_2 + w_3$
$w = 0.459 + 0.05 + 0.084 = 0.593$ k/ft
(8.65 kN/m)

(2) Live Load. Live load distribution factor,

$DF = \dfrac{S}{5.5} = 0.596$ (AASHTO (1996), Table 3.23.1)

Impact, $I = 0.3$

$\text{Factor} = (1 + I) \cdot 0.5 \cdot DF = 0.387$

b. *Exterior Girder.*

(3) Dead Load

Weight of the deck, $w = 0.14$ ksf (6.7 kPa)

Weight of the curb and rail (assumed), $w_{c\text{-}r} = 0.35$ k/ft (5.11 kN/m)

The dead load due to the weight of the slab was computed by considering a portion of the deck as a simple span with an overhang supported by the exterior and adjacent interior girders, as shown in Figure 22.

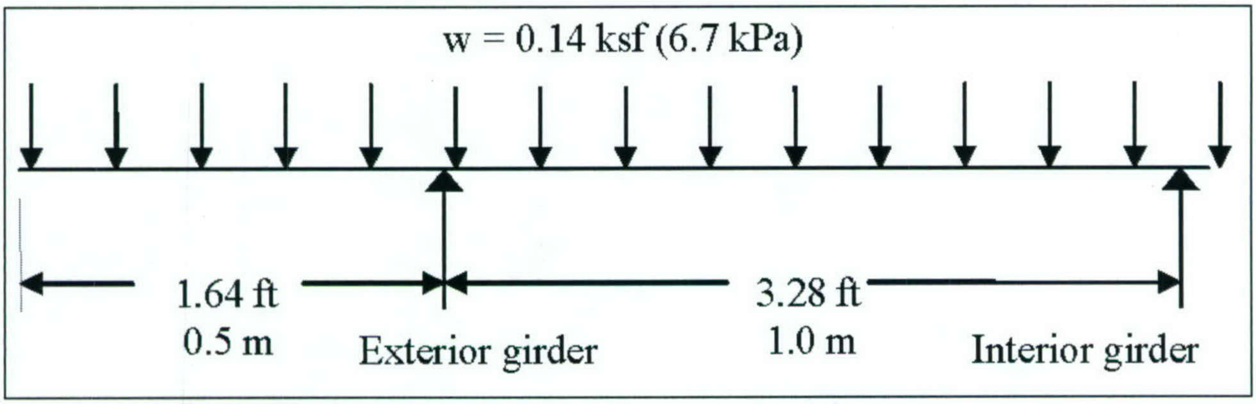


Figure 22. Exterior and adjacent interior girder supporting the deck (GDH 1953b)

Weight of the deck, $w_1 = 0.14\ ksf \cdot 4.92\ ft \cdot \dfrac{4.92\ ft/2}{3.28\ ft} = 0.517$ k/ft (7.543 kN/m)

Curb and railing load were distributed on 7 girders.

Curb and railing load, $w_2 = \dfrac{0.35}{7} = 0.050$ k/ft (0.73 kN/m)

Weight of Girder (increased by 50 percent), $w_3 = 0.084$ k/ft (1.226 kN/m)

Total dead load on girder, $w = w_1 + w_2 + w_3$

$w = 0.517 + 0.050 + 0.084 = 0.651$ k-ft (9.50 kN/m)

(4) Live Load

The portion of the live load on the exterior girder was calculated by the lever rule. The live load distribution factor for the exterior girder was calculated by applying to the girder the reaction of the wheel load obtained by assuming the flooring to act as a simple span between the exterior and the interior girders (AASHTO (1996), Section 3.23.2.3.1.2). The HET was placed at 2 ft (minimum allowable) from the curb to have the maximum possible load on the exterior girder. The location of the HET is shown in Figure 23.

$$R = \frac{1.28}{3.28} P = 0.39P$$

Minimum R = $\dfrac{S}{5.5} P = 0.596P$

(AASHTO (1996), Section 3.23.2.3.1.5)

Hence R = 0.596P, governs.

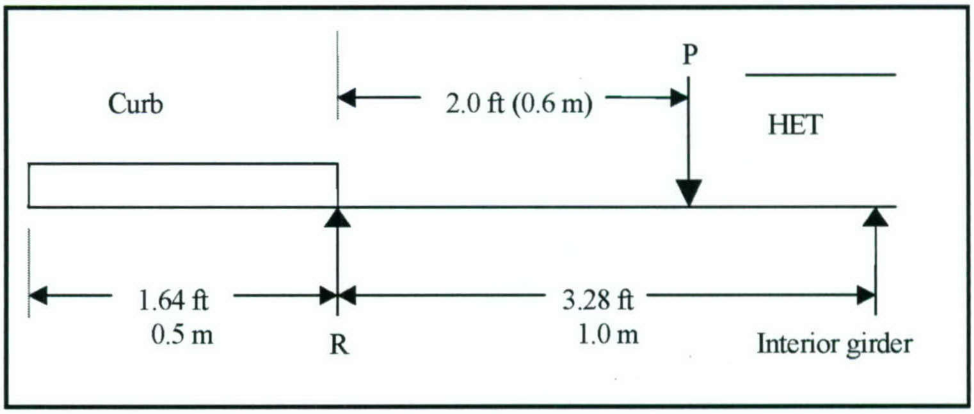


Figure 23. Placement of vehicle for the maximum load on the exterior girder (GDH 1953b)

$$\text{Impact, I} = \frac{50}{S+125} = \frac{50}{39.4\,ft+125} = 0.31 > 0.3 \quad \text{(Maximum allowable)}$$

Hence I = 0.3

$$\text{Factor} = (1+I)\cdot 0.5 \cdot R = 0.387$$

The factor, which would be used to modify the results obtained from SAP2000, was greater for the exterior girder than for interior girder. Therefore, the exterior girder was critical and was checked for loading.

c. Girder Check. The capacity of the girder was checked both at the midspan and at the support. Design forces obtained from SAP2000 at the midspan and at the support for the exterior girder, which was critical, were factored and tabulated, as shown in Tables 19 and 20.

Table 19
Maximum Moments for Exterior Girder

		Moment		
		Dead Load	**Live Load + Impact**	**Total**
Midspan	**kN-m**	55.5	280.0	335.4
	k-ft	40.9	206.5	247.4
Support	**kN-m**	115.6	299.0	414.6
	k-ft	85.24	220.56	305.8

Table 20 Maximum Shear Forces for Exterior Girder				
		Shear		
		Dead Load	Live Load + Impact	Total
Midspan	N	0	60.5	60.5
	kips	0	13.6	13.6
Support	N	59.1	182.8	241.8
	kips	13.28	41.09	54.37

(1) Girder at Midspan

Effective flange width, b_e : Minimum of followings; (AASHTO (1996), Section 10.38.3.1)

$$b_e = \frac{S}{4} = \frac{39.4\ \text{ft}}{4} = 9.8\ \text{ft}\ (3.0\ \text{m})$$

b_e = Girder spacing = 3.28 ft (1.0 m)

b_e = 12 times slab thickness = 9 ft (2.7 m)

Hence b_e =3.28 ft (1.0 m), governs.

Figure 24 shows the composite section at midspan.

Transformed width, $b_{tr} = \frac{b_e}{n} = \frac{3.28}{9} = 0.365\ \text{ft} = 4.4\ \text{in.}\ (0.11\ \text{m})$

(n: modular ratio)

Figure 25 shows the transformed section at midspan. The transformed section was used to find the capacity of the section.

The centroid of the section, y (calculated from bottom);

y = 14.72 in. (0.37 m)

Moment of inertia about the neutral axis, I_{NA};

I_{NA} = 2,903 in.^{4} (120,800 cm^4)

Where c: Distance between the neutral axis and point of interest, where the stress was checked.

Stress check at the bottom; (M is from Table 19)

$$f_s = \frac{M \cdot c}{I_{NA}} = \frac{247{,}400\ \text{lb} - \text{ft} \cdot 14.72\ \text{in.}}{2{,}903\ \text{in.}^4} \cdot 12\ \text{in./ft}$$

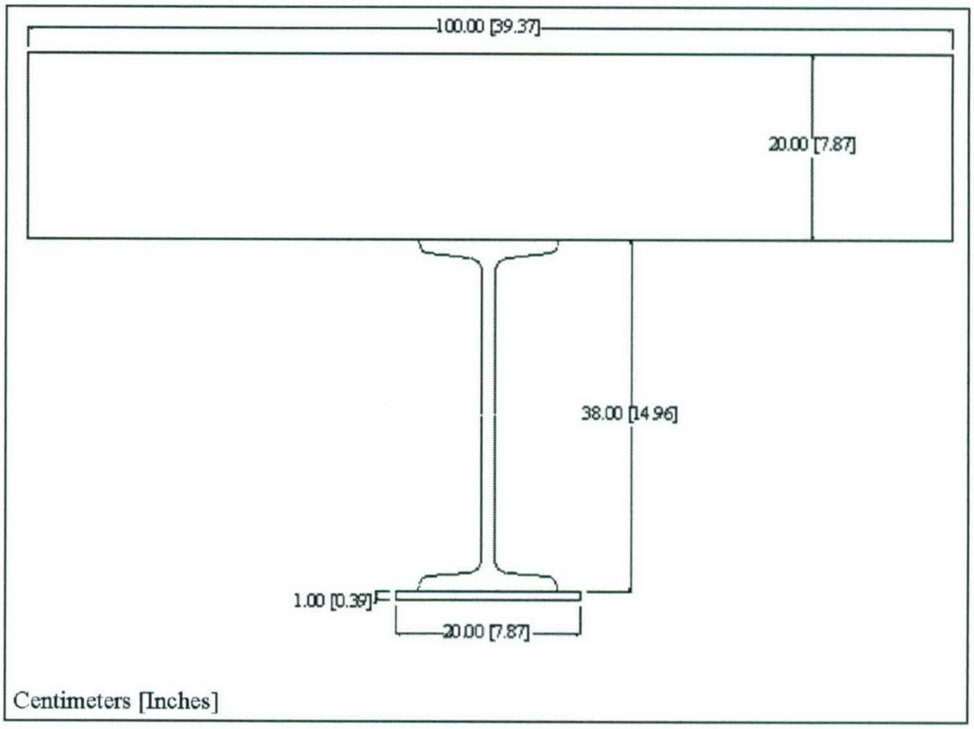


Figure 24. Composite section with cover plate at midspan (GDH 1953b)

$$f_s = 15{,}053 \text{ psi } (103.8 \text{ MPa}) < 20{,}000 \text{ psi } (137.9 \text{ MPa}) \qquad \text{OK.}$$

Stress check at the top of concrete slab; (M is from Table 19)

$$f_s = \frac{M \cdot c}{I_{NA}} = \frac{247{,}400 \text{ lb} - \text{ft} \cdot 8.5 \text{ in.}}{2{,}903 \text{ in.}^4 \cdot 9} \cdot 12 \text{ in./ft}$$

$$f_s = 965 \text{ psi } (6.7 \text{ MPa}) < 1{,}280 \text{ psi } (8.8 \text{ MPa}) \qquad \text{OK.}$$

(2) Girder at the Support

The slab was cracked (as it was in the tension zone) and was not added to the capacity of the section, shown in Figure 26. The rebars in the slab were assumed to be concentrated at their centroid, 13 cm (5.12 in) above the top of the cover plate.

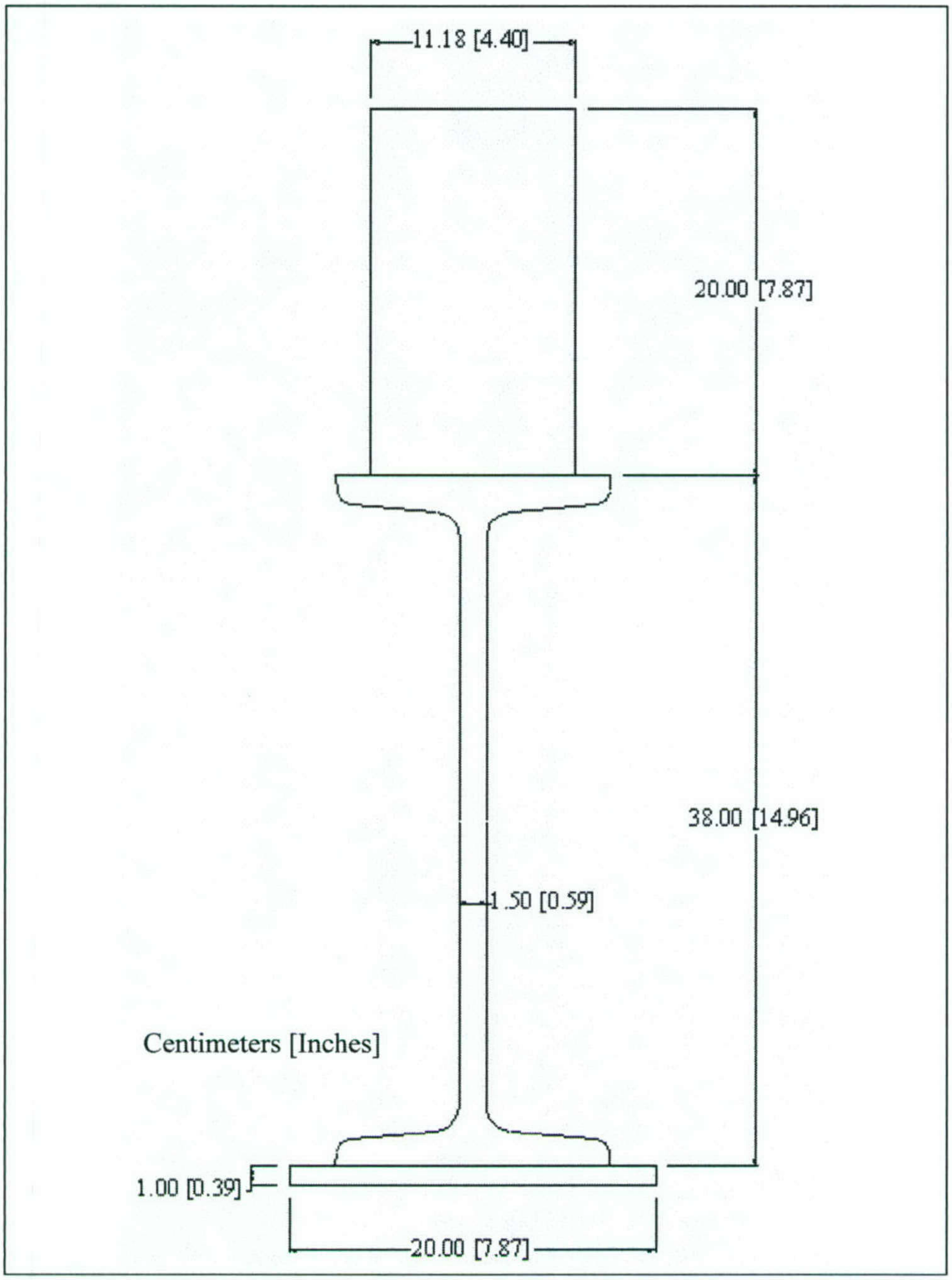


Figure 25. Transformed section at midspan (GDH 1953b)

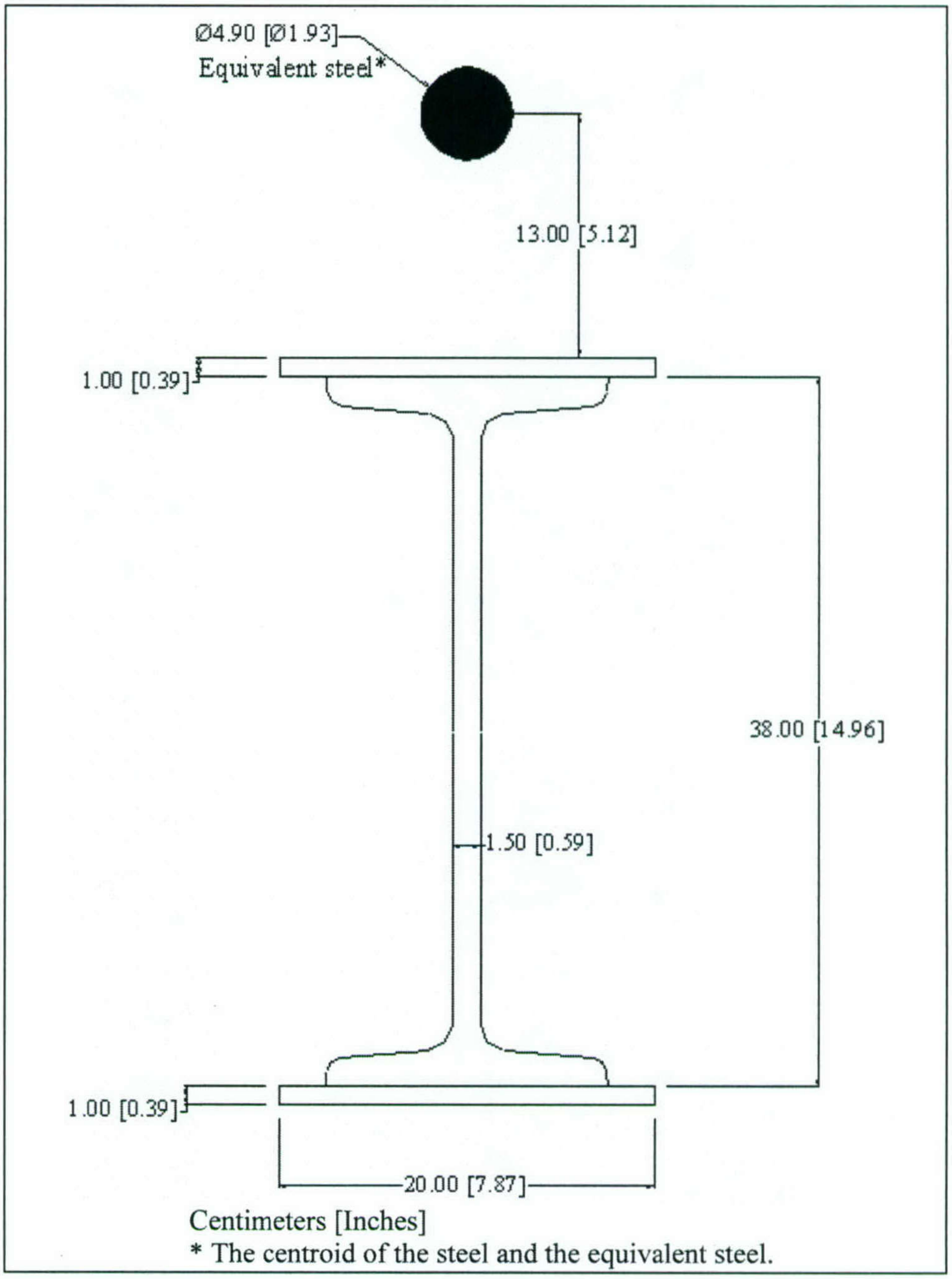


Figure 26. Section at the support (concrete slab was in tension zone) (GDH 1953b)

Centroid of the section, y (calculated from the bottom);

$y = 9.35$ in. (0.24 m)

Moment of inertia about neutral axis, I_{NA};

$I_{NA} = 1{,}376$ in.^{4} (57,270 cm^4)

Where c: Distance between the neutral axis and point of interest, where the stress was checked.

Stress check at the bottom; (M is from Table 19)

$$f_s = \frac{M \cdot c}{I_{NA}} = \frac{305{,}800\,\text{lb} - \text{ft} \cdot 9.3\,\text{in.}}{1{,}376\,\text{in.}^4} \cdot 12\,\text{in./ft} = 24{,}801 \text{ psi (171 MPa)}$$

f_s >20,000 psi (137.9 MPa) (allowable stress)

f_s <34,000 psi (234.4 MPa) (yield limit stress)

Stress check at the reinforcement; (M is from Table 19)

$$f_s = \frac{M \cdot c}{I_{NA}} = \frac{305{,}800\,\text{lb} - \text{ft} \cdot 11.51\,\text{in.}}{1{,}376\,\text{in.}^4} \cdot 12\,\text{in./ft} = 30{,}695 \text{ psi (211.6 MPa)}$$

f_s >20,000 psi (137.9 MPa) (allowable stress)

f_s <34,000 psi (234.4 MPa) (yield limit stress)

Although the stresses in the cover plate and slab reinforcement were greater than the allowable stresses, they were still under the yield limit stress. The stresses in the steel did not cause failure in the material and the steel did not yield.

(3) Shear Stress Check

Design shear stress, V_d = 54.37 kips (241.85 kN) (from Table 20)

Allowable shear stress, F_v

$$F_v = 0.33 \cdot F_y = 0.33 \cdot 34 = 11.22 \text{ ksi (77.36 MPa)}$$

Shear stress was assumed to be carried entirely by the web of the girder and it was assumed to be distributed uniformly in the web (AASHTO (1996), Section 10.38.5.2).

$$\text{Shear stress, } f_v = \frac{V}{d \cdot t_w} = \frac{54.37\,\text{kips}}{14.96\,\text{in.} \cdot 0.59\,\text{in.}} = 6.16 \text{ ksi (42.47 MPa)}$$

f_v < 11.22 ksi (77.36 MPa) OK.

Where
V: Maximum shear force (Table 20)
d : depth of I-Girder
t_w: thickness of web

Summary of Results

A composite steel I-Girder bridge, which has a maximum span of 12 m (39.37 ft), was analyzed for the HET. Results of these analyses are given below for each individual component of the bridge.

Slab

Thickness of the slab, main steel, and distribution and temperature steel used in the slab were satisfactory. The slab was adequate against shear failure.

Girder

The exterior girder was analyzed, as the stresses were higher than those in interior girder. Shear stress in the girder was found to be carried safely.

For bending stress at the midspan section, the girder was satisfactory. On the other hand, at the support section where the concrete slab was in tension zone and carries no stress, the girder did not satisfy the allowable stress requirements neither in the rebars at the top nor at the cover plate at the bottom. While both the top and the bottom stresses due to bending were greater than the allowable, they were smaller than the yield limit for the steel, ST37. The yield limit for ST37 steel is 34,000 psi (234.4 MPa). However, this value presents the *minimum* yield limit for that material. The actual yield strength of steel is typically higher than the minimum yield limit. The value of 34,000 psi (234.4 MPa) is not the average yield limit, but the minimum. Therefore, standard construction steel (such as ST37) would typically have more capacity than the specified minimum yield limit. The stress at the rebars, which was calculated as 30,600 psi (211 MPa), is close to the yield limit for the steel. However, it is smaller than the *minimum yield limit*, 34,000 psi (234.4 MPa).

To summarize, the capacity of the bridge according to the original drawings (without introducing any factors of safety) was acceptable as none of the stresses exceeded the minimum yield limits, although they were very high as described above.

6 Analysis of the Candir Bridge

Introduction

The Candir (or Hasanpasa) Bridge is a reinforced concrete T-Girder bridge. It was constructed in 1972. The bridge, which crosses the Candir River, is in northwestern Turkey and it is on the state highway connecting Inegol with Bozuyuk. The location of the bridge is marked with a star in Figure 27.

Figure 27. Location of the Candir Bridge on the traffic flow map

It is a two-lane bridge with a curb-to-curb width of 8.50 m (27.9 ft). The curb on each side has a width of 0.6 m (1.97 ft). The total width of the deck is 9.70 m (31.82 ft). The cross-section of the bridge is given in Figure 28 (GDH 1969).

The bridge has 7 simple spans; the maximum is 15.70 m (51.5 ft), with a total length of 113.50 m (372.38 ft). A view of one of the spans is shown in Figure 29 (GDH 1969).

A new two-lane bridge is presently under construction right next to Candir Bridge. This new bridge is a part of the government's highway improvement project, which would eventually double the number of lanes on crowded and critical highways. The construction, which started in 2001, is supposed to be finished at the end of 2003.

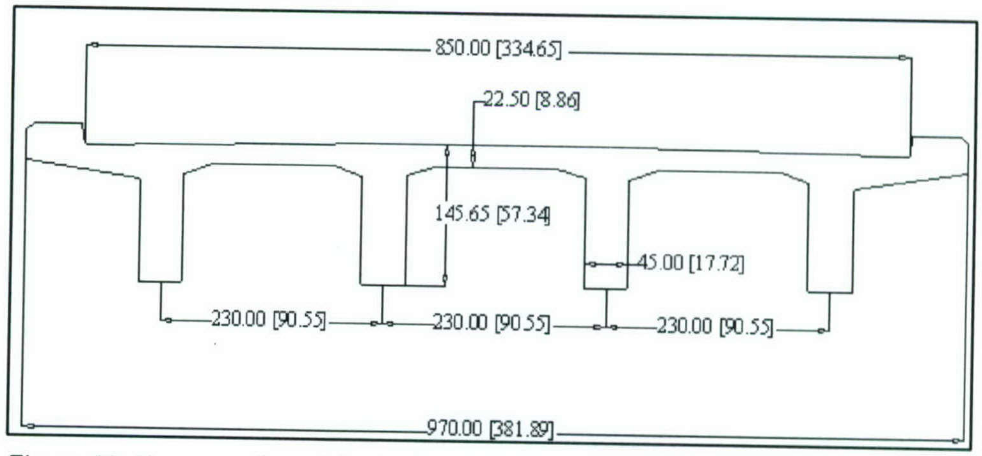


Figure 28. Cross-section of the bridge (GDH 1969) (centimeters [inches])

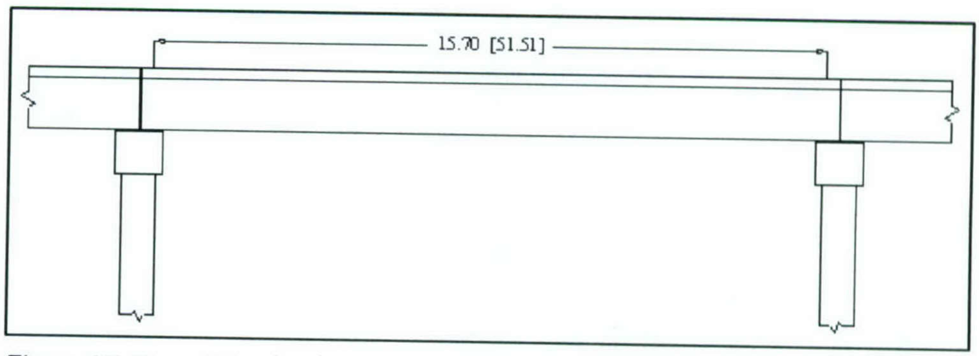


Figure 29. One of the simple spans (GDH 1969) (meters [feet])

Design live load for the new bridge is HS30 of TSRB, although it was HS20 for the current bridge (GDH 1969). Both bridges would be in service after the construction of second bridge is finished. Pictures of the new bridge are given in Appendix E.

The blueprints of the superstructure for the Candir Bridge were obtained from the GDH Ankara Headquarter. Analysis of the superstructure was done according to AASHTO (1996) using these design drawings. Several pictures of the bridge were taken and given in Appendix E.

The reinforcement used in the bridge construction was ST37, described in Chapter 2. Compression strength of concrete, f_c', was 22.1 MPa (3,200 psi) (GDH 1969). Design live load was HS20 of TSRB, which is 10 percent heavier than HS20-44 in AASHTO (1996) as described in Chapter 3.

Analysis of the Candir Bridge

Slab, exterior girder and interior girder were checked performing ASD approach prescribed in AASHTO (1996). Substructure of the bridge was not analyzed as the focus of the research was primarily on the superstructure.

The bridge was modeled in SAP2000 and internal moments were obtained from 2D analysis. Only the primary loads; dead load, live load and impact, were applied to the structure. Only one HET vehicle was assumed to be on the bridge at a time as prescribed in AASHTO (1996). Actual strengths of the sections were calculated without introducing any strength reduction factors, as described in Chapter 4. Loads were not modified by any factors.

Slab

a. *Effective Span.* Effective span, S, is the clear span between T-Girders

(AASHTO (1996), Section 3.24.1.2a)

$S = 72.83$ in. $= 6.069$ ft (1.85 m)

b. *Dead Load.* Dead load of the slab is calculated in terms of pressure (ksf).

Self weight of slab, w_1: $w_1 = t \cdot \gamma$

where t: thickness of slab in feet = 8.86 in. (0.23 m)

γ: Unit weight of concrete

$$w_1 = \frac{8.86\text{ in.}}{12\text{ in./ft}} \cdot 0.150\text{ kcf} = 0.1107\text{ ksf (5.3 kPa)}$$

Wearing surface, assumed, w_2: $w_2 = 0.019$ ksf (0.91 kPa)

Total dead load, w: $w = w_1 + w_2 = 0.1107 + 0.019 = 0.129$ ksf (6.21 kPa)

Dead load moment, M_D:

$$M_D = \pm\frac{w \cdot S^2}{10} = \frac{0.1297\text{ ksf} \cdot (6.069\text{ ft})^2}{10} = 0.478\text{ k} - \text{ft (0.648 kN-m)}$$

c. *Live Load.* Live load moment at midspan, M_L, is calculated according to the formula given in AASHTO (1996), Section 3.24.3.1 using the rear wheel of HET as the loading.

Rear wheel load of HET, $P_{HET} = 15.7$ kips (69.8 kN)

$$M_L = 0.8 \cdot \frac{S+2}{32} \cdot P_{HET} = 0.8 \cdot \frac{6.069\text{ft}+2}{32} \cdot 15.7\text{ kips} = 3.167\text{ k-ft (4.294 kN-m)}$$

Impact Factor, I: (AASHTO (1996), Section 3.8.2.1)

$$I \frac{50}{S+125} = \frac{50}{6.069\text{ ft} + 125} = 0.38 > 0.3 \qquad \text{(Maximum allowable)}$$

Hence I = 0.3

$M_{L+I} = (1+I)\cdot M_L = (1+0.3)\cdot 3.167\text{k}-\text{ft} = 4.117\,\text{k-ft}$ (5.582 kN-m)

Total design moment, $M_T = M_D + M_{L+I}$

M_T = 0.478 k-ft + 4.117 k-ft = 4.595 k-ft (6.23 kN-m)

d. Main Steel. Yield stress for ST37 steel was given in Turkish Standards, TS-648 (TSI 1980).

f_y: 34,000 psi (234.4 MPa)

Allowable stress for ST37 steel was given in Turkish Standards, TS-648 (TSI 1980).

f_s: 20,000 psi (137.9 MPa)

Allowable stress for concrete is 0.4 times of the concrete nominal strength (f_c').

$f_c = 0.4\cdot f_c' = 0.4\cdot 3{,}200\ psi = 1{,}280$ psi (8.8 MPa)

Modular ratio (n), the ratio of the modulus of elasticity of steel to that of concrete, was given in AASHTO (1996), Section 10.38.1.3. Modular ratio is dependent on the concrete nominal strength (f_c').

n = 9 for f_c'=3,200 psi (22.1 MPa)

The design coefficients k (neutral axis factor), and j (lever-arm factor) were calculated.

$$k = \frac{n}{n+\frac{f_s}{f_c}} = \frac{9}{9+\frac{20{,}000\,psi}{1{,}280\,psi}} = 0.365$$

$$j = 1 - \frac{k}{3} = 0.878$$

The distance from the concrete surface and the center of the rebars was 3 cm (1.22 in.) (GDH 1969). Therefore the depth of the slab (d) was calculated as,

$d = t - 1.22 = 7.64$ in.

Required slab depth, d_r:

$$d_r = \sqrt{\frac{M_T}{0.5\cdot f_c\cdot k\cdot j}} = \sqrt{\frac{4{,}595\text{ lb}-\text{ft}}{0.5\cdot 1{,}280\text{ psi}\cdot 0.365\cdot 0.878}}$$

$d_r = 4.74\ \text{in.}\ (0.12\ \text{m}) < 7.64\ \text{in.}\ (0.19\text{m})$ OK.

Minimum slab depth, d_{min}: (AASHTO (1996), Table 8.9.2)

$$d_{min} = \frac{S+10}{30} = \frac{6.069\ \text{ft} + 10}{30} \cdot 12\ \text{in./ft} = 6.43\ \text{in.}\ (0.16\ \text{m}) < 7.64\ \text{in.}\ (0.19\ \text{m})\ \text{OK.}$$

Supplied main steel = 0.534 in.^{2} per foot of slab (GDH 1969).

Required steel, A_s:

$$A_s = \frac{M_T}{f_s \cdot j \cdot d} = \frac{4{,}595\ \text{lb}-\text{ft}}{20{,}000 \cdot 0.878 \cdot 7.64} \cdot 12\ \text{in./ft}$$

$A_s = 0.41\ \text{in.}^2\ (2.65\ \text{cm}^2) < 0.534\ \text{in.}^2\ (3.445\ \text{cm}^2)$ OK.

e. Distribution Steel. (AASHTO (1996), Section 3.24.10.2)

Supplied distribution steel = 0.214 in.^{2} (1.38 cm^2) per foot of slab (GDH 1969).

$$\text{Percentage} = \frac{220}{\sqrt{S}} = \frac{220}{\sqrt{6.069}} = 89\ \% > 67\ \%\ \text{(Maximum allowable)}$$

The amount of distribution steel can be at most 67 percent of the supplied main steel. Therefore the required distribution steel is,

$\text{Required steel} = 0.67 \cdot 0.411 = 0.275\ \text{in.}^2\ (1.77\ \text{cm}^2)$
$> 0.214\ \text{in.}^2\ (1.38\ \text{cm}^2)$ NOT OK.

In AASHTO (1996), Section 3.24.10.1, the distribution steel is defined as the reinforcement placed transverse to the main steel reinforcements in the bottom of the slab to provide for the lateral distribution of the concentrated live loads. This much reinforcement would be O.K. as the total load of HET is spread on more tires than the AASHTO (1996) standard trucks (either H or HS).

f. Temperature Steel. According to AASHTO (1996), Section 8.20, the required temperature steel per foot of slab is constant and equal to 0.125 in.^{2} (0.806 cm^2).

Supplied temperature steel = 0.142 in.^{2} (0.916 cm^2) per foot of slab (GDH 1969).
Required steel = 0.125 in.^{2} (0.806 cm^2) < 0.142 in.^{2} (0.916 cm^2) OK.

g. Column Punching. According to AASHTO (1996), Section 8.15.5.6.1, the slab should be checked for punching shear in the vicinity of *concentrated loads* and reactions. The punching of the column into the slab was neglected, since the columns of the bridge have a column cap that supports the girders,

which in turn support the slab. This is shown in the Figures E5, E6, and E9 in Appendix E.

h. Tire Punching. If the load applied to the slab by a tire was greater than the punching shear capacity of the slab, the tire would penetrate ('punch') through the slab. The punching shear of a tire of the HET was checked according to AASHTO (1996), Section 8.15.5.6. According to AASHTO (1996), Section 3.30, the contact area of the tire, A, is:

$$A = 0.01 \cdot P$$

where P = Load on the tire (lb)

The most critical tire was one of the rear tires, for which the load was 7,850 lb (34.92 kN). For this load, the contact area was calculated as 78.5 in.^{2} (506.5 cm^2), using the equation given above. The contact area is given as a rectangle, for which the ratio of the long side to the short side is 2.5. Therefore the rectangle was 14 in. $\times$ 5.6 in. (0.36 m $\times$ 0.14 m). Punching stress is calculated by;

$$v = \frac{V}{b_0 \cdot d} \qquad \text{(AASHTO (1996), Equation 8-12)}$$

where,

v = Shear stress

b_0 : Perimeter of the critical section (AASHTO (1996), Section 8.15.5.6.1b)

d : Depth of the section

$$v_c = (0.8 + \frac{2}{\beta_c})\sqrt{f_c'} < 1.8\sqrt{f_c'} \qquad \text{(AASHTO (1996), Equation 8-13)}$$

where v_c : Punching shear stress capacity

f_c': Nominal strength of concrete

Therefore, punching shear capacity, V_c, is:

$$V_c = v_c \cdot b_0 \cdot d = 48.6 \text{ kips (216.2 kN)}$$

Tire Load = 7.85 kips (34.92 kN) < 48.6 kips (216.2 kN) O.K.

Girders

Minimum depth of T-Girder = 0.07S (AASHTO (1996), Table 8.9.2)

$d_{min} = 0.07 \cdot (51.51\,\text{ft}) \cdot 12\,\text{in./ft} = 43.27$ in. (1.1 m) < 55.7 in. (1.4 m) OK.

Curb and railing load was assumed to be distributed equally to all stringers according to AASHTO (1996), Section 3.23.2.3.1.1).

a. Interior Girder.

(1) Dead Load

Weight of the slab was calculated earlier. The spacing of the girders was equal to 7.546 ft (2.3 m). Therefore the weight of the slab over one girder was calculated as,

Weight of the deck, $w_1 = 0.1297\,\text{ksf} \cdot 7.546\,\text{ft} = 0.979$ k/ft (14.28 kN/m)

Curb and the railing load were distributed to 4 girders.

Curb and railing load, $w_2 = \dfrac{0.600}{4} = 0.150$ k/ft (2.189 kN/m)

Weight of the T-Girder Stem, $w_3 = 0.944$ k/ft (13.773 kN/m)

Total load on the girder, $w = w_1 + w_2 + w_3$

$w = 0.979 + 0.150 + 0.944 = 2.073$ k/ft (30.25 kN/m)

Dead load moment, $M_D = \dfrac{w \cdot L^2}{8} = 687.5$ k-ft (932.1 kN-m)

Dead load shear, $V_D = 0.5 \cdot w \cdot L = 53.4$ kips (237.5 kN)

(2) Live Load

Maximum live load moment, M_L:

M_L=1,347 k-ft (1,826.3 kN-m) (calculated in SAP2000 for HET)

Live load distribution factor, $DF = \dfrac{S}{6} = 1.258$

(AASHTO (1996), Table 3.23.1)

Impact, $I = \dfrac{50}{L+125} = \dfrac{50}{51.51\,ft + 125} = 0.283$

$M_{L+I} = (1 + I) \cdot 0.5 \cdot DF \cdot M_L$

$M_{L+I} = (1 + 0.283) \cdot 0.5 \cdot 1.258 \cdot 1{,}347\,k - ft = 1{,}087$ k-ft (1,473.8 kN-m)

Total Moment,

$M_T = M_D + M_{L+I}$

M_T = 687.5 k-ft + 1,087 k-ft = 1,774.5 k-ft (2,405.9 kN-m)

Maximum live load shear calculation

(AASHTO (1996), Section 3.23.1)

Rear wheel of a HET vehicle was positioned on an interior girder. A second vehicle was assumed to be next to the first one, 4 ft apart. Four (4) ft is the minimum distance (to get the maximum effect) according to AASHTO (1996). This positioning, shown in Figure 30, gave the maximum load distribution factor.

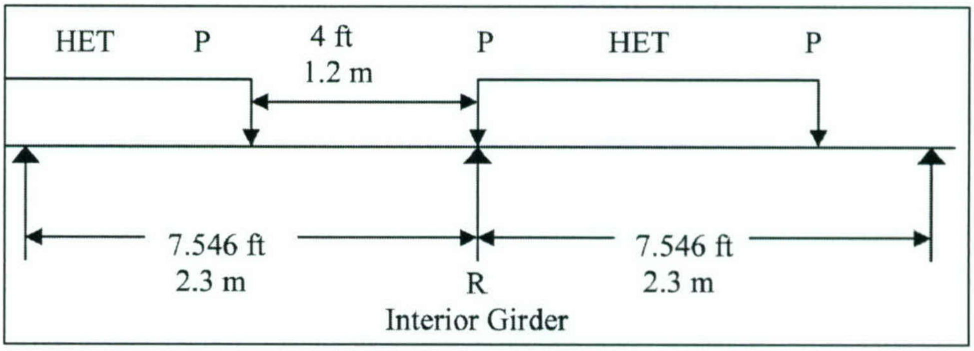


Figure 30. HET positioning on the deck for critical case (GDH 1969)

Distribution factor was calculated assuming the deck was simply supported on the girders.

$$R = P + \frac{7.546\,ft - 6\,ft}{7.546\,ft} \cdot P + \frac{7.546\,ft - 4\,ft}{7.546\,ft} \cdot P = 1.675\,P$$

The load on the interior girder due to rear axle, R:
$R = 1.675 \cdot P = 1.675 \cdot 15.7\ kips = 26.3\ kips$ (117 kN)

To find the maximum shear, HET vehicle was positioned on the span such that the rear axle rests on the support, as shown in Figure 31.

V_L= Shear due to rear load + DF*(Shear due to other axles)

where DF: Live load distribution factor

$$V_L = 26.3\ k + \frac{1.258}{51.51\,ft} \cdot \begin{bmatrix} (11.13 \cdot 2.64 + 11.05 \cdot 7.6 + 9.95 \cdot 12.6 + 13.5 \cdot 27.74 \\ +14.83 \cdot 33.7 + 14 \cdot 39.62 + 14.425 \cdot 45.56)\ kips - ft \end{bmatrix}$$

V_L = 83.1 kips (369.6 kN)

$$V_{L+I} = (1 + I) \cdot V_L$$

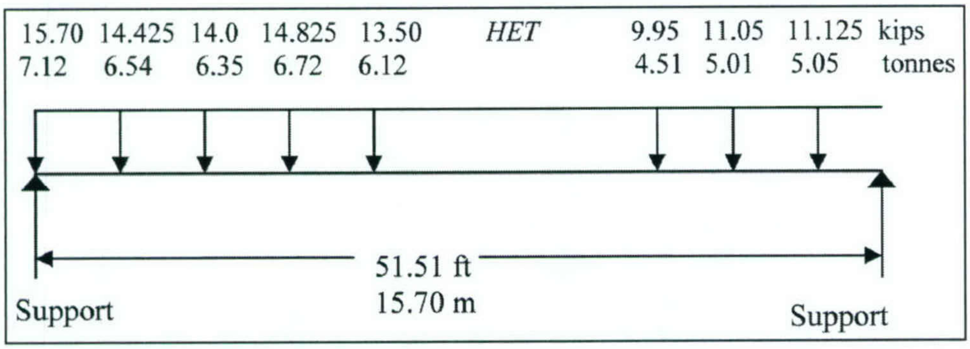


Figure 31. Positioning of HET for maximum shear force at support (GDH 1969)

$$V_{L+I} = (1 + 0.283) \cdot 83.1 \text{ kips} = 106.6 \text{ kips} \,(474.2 \text{ kN})$$

Total shear, V_T:

$$V_T = V_D + V_{L+I}$$

$$V_T = 53.4 + 106.6 = 160 \text{ kips} \,(711.7 \text{ kN})$$

(3) Check Web Area for Shear

Allowable shear stress in concrete,

$v_c = 0.95\sqrt{f_c'}$ (AASHTO (1996), Section 8.15.5.2.1)

Allowable shear stress in the rebars,

$v_s = 4\sqrt{f_c'}$ (AASHTO (1996), Section 8.15.5.3.9)

Maximum allowable shear for the section, v;

$$v = 0.95\sqrt{f_c'} + 4\sqrt{f_c'} = 4.95\sqrt{f_c'} = 4.95\sqrt{3{,}200} = 280 \text{ psi} \,(1.9 \text{ MPa})$$

Web area = $b_w \cdot d = 17.72 \text{ in.} \cdot 52.5 \text{ in.} = 930 \text{ in.}^2 \;(0.6 \text{ m}^2)$
(GDH 1969).

$$\text{Web area required} = \frac{V_T}{v} = \frac{160 \text{ kips}}{0.280 \text{ ksi}} = 572 \text{ in.}^2 \;(0.37 \text{ m}^2) < 930 \text{ in.}^2 \;(0.6 \text{ m}^2) \text{ O.K.}$$

(4) Check Moment Capacity of the Interior Girder

According to AASHTO (1996), Section 8.10.1.1 effective flange width, b_e, is the minimum of the following:

- $b_e = \dfrac{\textit{Span Length}}{4} = \dfrac{51.51\,\textit{ft}}{4} = 12.9 \text{ ft} \,(3.9 \text{ m})$

- b_e = Girder spacing = 7.546 ft = 90.55 in (2.3 m), governs.

- b_e = 12 times slab thickness+ web width = 10.54 ft (3.21 m)

Figure 32 shows the T-Girder beam section with effective flange and the tension rebars. Neutral axis (N.A.) was found to be at 12.58 in (0.32 m) from top of the flange. It was assumed that force in the compression block acts at one third of N.A.

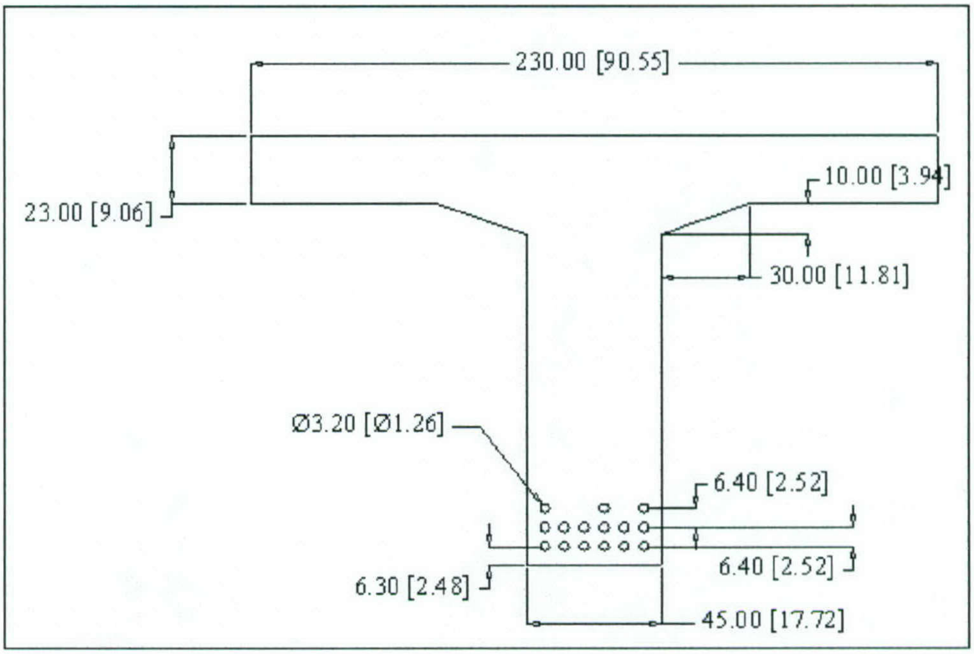


Figure 32. T-Girder section and tension rebars (GDH 1969) (centimeters [inches])

Stress in tension steel, $f_s = \dfrac{M_T}{A_s(d - \dfrac{kd}{3})}$

where
M_T: Total moment
A_s: Area of tension steel
d: Depth of the beam
kd: Neutral axis from top flange

$$f_s = \frac{1{,}774.5 \text{ k} - \text{ft} \cdot 12 \dfrac{\text{in.}}{\text{ft}} \cdot 1{,}000 \dfrac{\text{lb}}{\text{k}}}{18.7 \text{ in.}^2 \cdot (52.5 \text{ in.} - \dfrac{12.58 \text{ in.}}{3})} = 23{,}570 \text{ psi } (162.5 \text{ MPa})$$

f_s >20,000 psi (137.9 MPa) (allowable stress)

f_s <34,000 psi (234.4 MPa) (yield limit stress)

(5) Check Shear Reinforcement

According to AASHTO (1996), Equation 8-4, shear carried by concrete, v_c, is calculated at "d" distance away from face of the support, where "d" is the depth of the beam.

$$v_c = 0.9\sqrt{f_c'} + 1{,}100\left(\frac{A_s}{b_w \cdot d}\right)\cdot\left(\frac{V\cdot d}{M}\right) \le 1.6\sqrt{f_c'} \quad \text{(AASHTO (1996), Equation 8-4)}$$

where f_c': Compressive strength of concrete

A_s: Area of steel

b_w: Web width

d: Depth of the beam

V: Shear at "d" distance away from face of support

M: Moment at "d" distance away from face of support

To be able to use AASHTO (1996), Equation 8-4, it requires that:

$$\frac{V\cdot d}{M} < 1.0$$

$$\frac{V\cdot d}{M} = \frac{124\ \text{kips}\cdot 52.5\ \text{in.}}{8{,}172\ \text{kips} - \text{in.}} = 0.797 < 1.0 \qquad \text{O.K}$$

$$v_c = 0.9\sqrt{3{,}200\ \text{psi}} + 1{,}100\cdot\left(\frac{18.7\ \text{in.}^2}{17.72\ \text{in.}\cdot 52.5\ \text{in.}}\right)\cdot(0.797) = 68.5\ \text{psi} \le 1.6\sqrt{f_c'} = 90.5\ \text{psi}$$

Therefore v_c= 85.5 psi (0.6 MPa)

Shear stress carried by steel, v_s:

$$v_s = \frac{V}{b_w \cdot d} - v_c = \frac{124{,}000\ \text{lb}}{17.72\ \text{in.}\cdot 52.5\ \text{in.}} - 68.5\ \text{psi} = 64.8\ \text{psi} \quad (0.4\ \text{MPa})$$

Area of supplied stirrup, A_v, and spacing, s, at the girder:

$A_v = 0.35\ \text{in.}^2\ (2.26\ \text{cm}^2)$

s = 9.84 in. (0.25 m)

Check stress in the stirrup, f_s:

$$f_s = \frac{v_s \cdot b_w \cdot s}{A_v}$$

$$f_s = \frac{64.8\ \text{psi} \cdot 17.72\ \text{in.} \cdot 9.84\ \text{in.}}{0.35\ \text{in.}^2} = 32{,}282\ \text{psi}\ (222.6\ \text{MPa})$$

f_s >20,000 psi (137.9 MPa) (allowable stress)

f_s <34,000 psi (234.4 MPa) (yield limit stress)

The stress in the stirrup was very close to the minimum yield limit; however the rebars were not expected to yield at this level of stress as the minimum yield strength of the steel is 34,000 psi (234.4 MPa). Moreover, to simplify the analysis the HET load, which was spread over 28 tires, was assumed to be spread over 18 tires (2 tires per axle). This was a conservative assumption. The stresses due to the load spread over 28 tires would have been smaller than these stresses. Therefore the section capacity was acceptable as the stress was smaller than the minimum yield limit and considering the conservative approach used.

b. *Exterior Girder.*

(1) Dead Load

Total load on girder, w = 2.073 k-ft (2.811 kN-m)(same as interior girder)

$$M_D = \frac{w \cdot L^2}{8} = 687.5\ \text{k-ft}\ (932.1\ \text{kN-m})$$

$$V_D = 0.5 \cdot w \cdot L = 53.4\ \text{kips}\ (237.5\ \text{kN})$$

(2) Live Load

The portion of the live load on the exterior girder was calculated by lever rule. The live load distribution factor for the exterior girder was calculated by applying to the girder the reaction of the wheel load obtained by assuming the flooring to act as a simple span between the exterior and the interior girder (AASHTO (1996), Section 3.23.2.3.1.2). The HET was placed 2 ft (minimum allowable) from the curb to have the maximum possible load on the exterior girder. The placement of HET is shown in Figure 33.

$$R = 1.361\,P$$

Minimum $R = \frac{S}{6} P = 1.26\ P$ (AASHTO (1996), Section 3.23.2.3.1.5)

Hence R = 1.361 P, governs.

Impact, I = 0.283

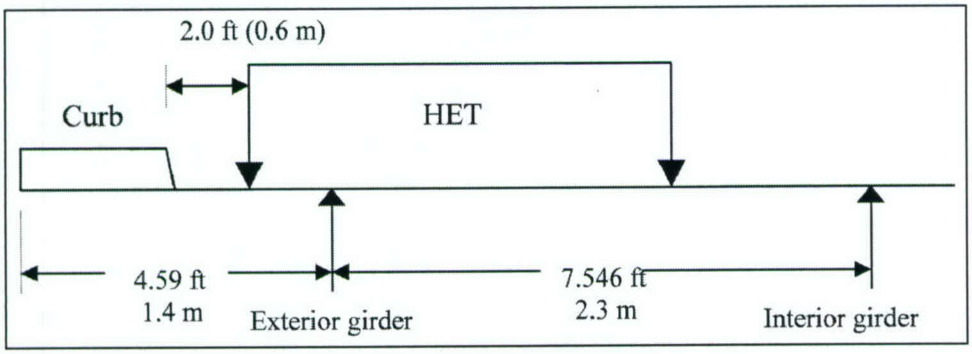


Figure 33. Placement of vehicle for the maximum load on the exterior girder

$M_{L+I} = (1 + I) \cdot 0.5 \cdot R \cdot M_L = 1{,}176.0$ k-ft (1,594.4 kN-m)

$M_{L+I} = (1 + 0.283) \cdot 0.5 \cdot 1.361 \cdot 1{,}347 \text{ k-ft} = 1{,}176.0$ k-ft (1,594.4 kN-m)

Total Moment,

$M_T = M_D + M_{L+I}$

$M_T = 687.5$ k-ft + 1,176 k-ft = 1,863.5 k-ft (2,526.6 kN-m)

Maximum total shear, V_T: (refer to Figure 6.4)

$VL = DF \cdot (\text{Shear due to rear load} + \text{Shear due to other axles})$

where DF: Live load distribution factor

$$V_L = 1.361 \cdot 15.7\ kips + \frac{1.361}{51.51 ft} \cdot (11.125 \cdot 2.64 + 11.05 \cdot 7.64 + 9.95 \cdot 12.64 + 13.5 \cdot 27.74 + 14.825 \cdot 33.68 + 14 \cdot 39.62 + 14.425 \cdot 45.56)\ kips - ft$$

V_L= 82.8 kips (368.3 kN)

$V_{L+I} = (1 + I) \cdot V_L$

$V_{L+I} = (1 + 0.283) \cdot 82.8 \text{ kips} = 106.2 \text{ kips} (472.4 \text{ kN})$

Total shear, V_T:

V_T= Shear due to dead load + Shear due to live load + Impact

$V_T = 53.4 \text{ kips} + 106.2 \text{ kips} = 159.6 \text{ kips} (710 \text{ kN})$

(3) Check Moment Capacity of the Exterior Girder

Exterior girder was assumed to have same capacity as the interior girder.

Check stress in steel, f_s:

$$f_s = \frac{M_T}{A_s(d - \frac{kd}{3})}$$

where M_T: Total moment

A_s: Area of tension steel

d: depth of the beam

kd: Neutral axis from top flange

$$f_s = \frac{1{,}863.5\,\text{k} - \text{ft} \cdot 12\,\frac{\text{in.}}{\text{ft}} \cdot 1{,}000\,\frac{\text{lb}}{\text{k}}}{18.7\,\text{in.}^2 \cdot (52.5\,\text{in.} - \frac{12.58\,\text{in.}}{3})} = 24{,}755\text{ psi} \;\; (170.7\text{ MPa})$$

f_s >20,000 psi (137.9 MPa) (allowable stress)

f_s <34,000 psi (234.4 MPa) (yield limit stress)

The calculated stress was acceptable as it is smaller than the minimum yield limit and considering the conservative approach used (HET was assumed to have 18 tires instead of 28).

(4) Check Shear Reinforcement

It was assumed that shear stress carried by steel was same as the interior girder. Therefore the stress in the exterior girders stirrups was same as the stress in the interior girders stirrup. See Page 76 of this report "(5) Check Shear Reinforcement."

Summary of Results

A reinforced concrete T-Girder bridge, which has a maximum span of 15.7 m (51.51 ft), was analyzed according to AASHTO (1996) using the HET as the live load.

Slab

Thickness of the slab, main steel, and temperature steel supplied in the slab are satisfactory according to ASD; however, the distribution steel is 22 percent less than what is required. The distribution steel helps the slab distribute the concentrated loads effectively to the girders. This much distribution steel would

be O.K. considering the conservative approach used (HET was assumed to have 18 tires instead of 28, which resulted in more concentrated loads than the actual case). The section was adequate against shear failure.

Girder

Interior and exterior girders are analyzed according to ASD. Maximum bending stress is found to be 23.6 ksi (162.7 MPa) in interior girder and 24.8 ksi (171 MPa) in exterior girder. These are higher than allowable stress (20,000 psi) for ST37 steel; however, they are still less than yield limit of the steel (34,000 psi). Therefore girders are safe according to bending stress without considering any factors of safety.

Tensile stress carried by the stirrups was calculated as 32.3 ksi (222.7 MPa) in both girders. This is more than 60 percent higher than allowable stress (20,000 psi) but still less than the minimum yield limit of ST37 steel. Although the stress in the stirrups was very critical and they were almost yielded it could be said that they are safe as the materials did not yield.

7 Summary and Conclusions

Introduction

Turkish design specifications and construction materials were compared with American specifications and materials. Three types of bridges were analyzed according to AASHTO (1996) using the HET as the live load on the bridges. Only the primary loads, dead load, live load and impact, were considered. The analysis did not include any modification for deterioration, damages, and aging of the bridges. The achievements and the results are summarized below.

Comparison of Specifications

The Turkish highway bridge design division currently uses the TSRB for the loading and geometric criteria only and AASHTO (1996) for all other requirements and design methods (Japan International Cooperation Agency 1996). In fact TSRB was adopted from AASHTO (1977). Hence there was no major difference between these two specifications regarding design methodology and requirements. There were some minor differences, which were pointed out in Chapter 3, due to conversion from U.S. customary units to SI units.

Currently the GDH is translating the latest AASHTO (1998) edition into Turkish, which will be used soon in bridge design (GDH 2003). Moreover Engineering Work Criteria Report (EWCR), published by the GDH in 1997, states that "Unless otherwise stated, all the bridges shall be designed according to the latest edition of AASHTO (1996) or AASHTO (1977)" (GDH 1997).

The most significant difference between Turkish and American live loading was in what the number coming after H or HS represents in two standards. In TSRB that number shows the weight of the truck in *metric tons*, but in AASHTO (1996) it shows the weight of the truck in *English (short) tons*. This also applies for lane loading. This results in a 10 percent heavier loads used for bridge design in Turkey. *Therefore, HS20 in TSRB is **10** and HS30 in TSRB is **65** percent heavier than HS20-44 in AASHTO.*

In general, bridges are designed to higher loads in Turkey. Although AASHTO (1996) is being used, the GDH requires higher design live loads. Currently, the design load for highway bridges is HS20 or HS30, depending on the traffic volume of the highway (GDH 1997).

For expressway (equivalent to interstates) bridges design load is HS30 or Alternate Military Loading (AML-36,000 kg or 80 kips), described in Chapter 3. These bridges are also checked with Type-A (136,000 kg or 300 kips) and Type-B (154,000 kg or 340 kips) vehicles. Figures of the design vehicles are given in Appendix B. In the U.S., although AASHTO (1996) has not introduced a heavy commercial hauler loading yet, each State's Department of Transportation developed its own design truck for this purpose.

The engineers in the bridge design division of the GDH had been to U.S. for training in several Departments of Transportations (DOT). According to Ms. Fatma Sahin, engineer in Bridge Design Division of the GDH in Ankara, the GDH has strong connections with U.S. DOT's, such as Pennsylvania Department of Transportation (PENNDOT) and California Department of Transportation (Caltrans). Currently the GDH uses PENNDOT's bridge design software for prestressed concrete bridge design. While using the software the design engineer increases the live load 10 percent because of the reason described above.

Analysis Results

The blueprints of Birecik Bridge could not be obtained from the GDH as the bridge is located in a 'security zone' for the Government. Therefore, the bridge was analyzed using the external dimensions and assuming that the minimum reinforcement according to ACI 318 (ACI 2002) was present.

The analysis was done in two parts. First, the slab was analyzed. It was safe. In the second part, the bridge was modeled in 2-D in SAP2000. The analysis of 2-D model in SAP2000 and in PCACOL showed that all the members, but the columns were satisfactory assuming minimum amount of steel at the sections. The columns did not satisfy the requirements with the minimum steel (1 percent) assumption. Blueprints of a similar bridge, Dokuzdolambac II Bridge (GDH 1953a), were obtained from the GDH. The steel ratio in the columns of this bridge was assumed to be present in Birecik Bridge. Instead of 1 percent reinforcement 2.64 percent was assumed. The column was not safe after this assumption, but would be safe assuming 2.94 percent reinforcement, which is within the range of steel reinforcing in the Dokuzdolambac II Bridge (GDH 1953a).

The shear was critical in T-Girders, assuming the minimum steel for the stirrups according to ACI 318 (ACI 2002) was used. Then a new assumption was made according to the blueprints of Dokuzdolambac II Bridge (GDH 1953a). With this assumption the stress in the stirrups were in the acceptable range.

The slab of the steel composite I-Girder bridge was safe. In the girder, the only critical stress was the bending stress at the support. It was more than the allowable stress, but still 10 percent less than the minimum yield limit of the steel, 34,000 psi. Therefore it was in the acceptable range.

The results of the analysis of the Candir Bridge showed that the slab was safe. The bending stresses were 20 percent higher than the allowable stresses, which was acceptable. The stress in the stirrups was around 32,000 psi

(220.6 MPa). Although critical, it was still 5 percent less than the minimum yield limit of steel, 34,000 psi. Therefore it was acceptable.

Conclusions of Analysis

The overall assessment for the Birecik Bridge (with the assumptions made in the analysis, given in Chapter 4) indicates that the columns of the bridge were critical and may not be adequate to support a HET vehicle safely. If the reinforcing ratio was assumed to be the average of the range seen in a similar bridge, the columns would be safe. For the I-Girder and the Candir Bridges, the highest stresses in the superstructures were at least 5 percent smaller than the minimum yield limit of steel, 34,000 psi. In actual case, this steel would typically yield at more than 34,000 psi as this value is the minimum requirement of the code. Moreover, assumed transverse geometry of HET resulted in higher distribution factors and higher stresses than the actual vehicle as it was assumed that the weight of HET was distributed over 18 tires instead of 28.

Further Research

Based on the findings of the research conducted in this project, the following additional studies are recommended:

a. This project has focused almost exclusively on the superstructure analysis of the bridges. The substructure (piers, abutments, bearings, piles, etc.) should be analyzed.

b. This project did not consider any possible damage, aging, environmental factors, or the possible poor workmanship. Present condition of the bridges should be investigated and the analysis should be modified to take these factors into account.

c. The bridges should be analyzed according to AASHTO-Load and Resistance Factor Design (LRFD) (AASHTO 1998) for overloading to examine the effect of different analysis methods on the results.

d. Some selected bridges in seismic zones should be analyzed according to AASHTO (1996), which is also used in bridge design in Turkey. These bridges will rate higher than those analyzed according to AASHTO (1996).

e. Different types of bridges should be analyzed. Especially stone arch bridges, some of which are more than 500 years old and still in service on provincial roads, should be checked for overloading.

f. Turkish bridge construction practices should be studied by conducting field surveys and site investigations.

References

American Association of State Highway and Transportation Officials. (1977). "Standard specifications for highway bridges," 12th ed., Washington, DC.

American Association of State Highway and Transportation Officials. (1996). "Standard specifications for highway bridges," 16th ed., 1996, Washington, DC.

American Association of State Highway and Transportation Officials. (1998). "LRFD Bridge Design Specification,"2nd ed., Washington, DC.

American Association of State Highway and Transportation Officials. (2003). "2000 Interim to Guide Specifications for Seismic Isolation Design," Washington, DC.

American Concrete Institute 318. (2002). "Building code requirements for structural concrete (ACI 318-02) and commentary (ACI 318R-02)," Detroit, MI.

American Society for Testing and Materials. (1996) "Standard specification for axle-steel deformed and plain bars for concrete reinforcement," Designation A 617-96, ASTM International.

American Society for Testing and Materials. (2002). "Standard specification for steel strand, uncoated seven-wire for prestressed concrete," Designation A 416/ A 416M-02, ASTM International.

AutoCAD 2002. (2001). "AutoCAD 2002"(computer program), 111 Mcubbus Oarjwatm Sab Rafaekm, CA 94903.

Computers and Structures Inc. (2002). "SAP2000 Non-Linear Version 8.1.2, Bridge Analysis, Analysis Reference Manual," Berkeley, California, July, 2002, Chapter 22.

Computers and Structures Inc. (2003a). "SAP2000 Non-Linear Version 8.1.2, Manuals: Introduction," Berkeley, CA.

Computers and Structures Inc. (2003b). "SAP2000 Non-Linear Version 8.1.2, Manuals: Software Verification Examples," Berkeley, California.

General Directorate of Highways. (1953a). "Blueprints, Dokuzdolambac II Bridge," Ankara, Republic of Turkey.

General Directorate of Highways. (1953b). "Blueprints, I-38 composite-continuous girder standard bridge types," Type No: 102/53, Ankara, Republic of Turkey.

General Directorate of Highways. (1953c). "Standard bridge types, 1953," Ankara, Republic of Turkey.

General Directorate of Highways. (1969). "Blueprints, Candir (Hasanpasa) Bridge," Type No: C-47/10, Ankara, Republic of Turkey.

General Directorate of Highways. (1982). "Technical specification for roadway bridges (TSRB)," Ankara, Republic of Turkey. Publication No: 207.

General Directorate of Highways. (1997). "Engineering works criteria report (EWCR)," Ankara, Republic of Turkey.

General Directorate of Highways. (2001). "Turkish highways 1950-2001," Ankara, Republic of Turkey.

General Directorate of Highways. (2003). "Interview with Ms. Fatma Sahin, Engineer, Bridge Design Division," Ankara, Republic of Turkey.

Japan International Cooperation Agency. (1996). "The study on the maintenance and rehabilitation of highway bridges in the Republic of Turkey," Final Report, *Main Text* 2(08). (12)

Menheere, Sebastiaan C.M., and, Spiro N., Pollalis. (1996). "Case studies on build operate transfer," Delft University of Technology, P.O. Box 5043, 2600 GA Delft, The Netherlands, 8.

Metz, Helen Chapin. (1996). Turkey: A Country Study, Washington: GPO for the Library of Congress.

Minor J., Woodward C. (2001). "Bridge analysis study for the military heavy equipment transporter system, phase II and phase III"; Final Report, New Mexico State University.

Portland Cement Association. (1999). "PCACOL Version 3.0" (computer program), Computer Programs Unit, Skokie, Illinois.

Republic of Turkey. "General Directorate of Highways Web Site." Retrieved October 2003. http://www.GDH.org.tr/index.htm.

Turkish Standards Institution. (1980). "Building code for steel structures," Designation TS 648, December, 1980.

Turkish Standards Institution. (1996). "Steel bars for concrete," Designation TS 708.

Turkish Standards Institution. (2000). "Requirements for design and construction of reinforced concrete structures," Designation TS 500.

Turkish State Railways. (2003). "Document No: B.11.2.DDY.0.63.00.03/3/2951, TCDD," Ankara, Turkey.

Appendix A
Sample Bridge Inventory Cards

GENERAL DIRECTORATE OF HIGHWAYS

Directorate of Bridge Maintenance and Science Committee

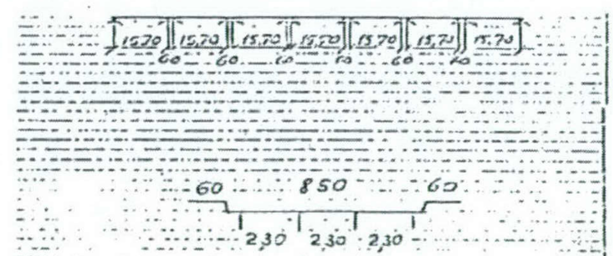
7581120 - BRIDGE INVENTORY FIELD CARD

BRIDGE SERIAL NO

BRIDGE	PROJECT NAME	CANDIR (HASANPAŞA)
	LOCAL NAME	
VEHICLE COUNTER VALUES	HIGHWAY SECTION STARTING POINT AND km	
	BRIDGE LOCATION km	
	HIGHWAY SECTION END km	
	IMPORTANT POINTS km	
LOCATION OF BRIDGE	DIVISION NO.	14
	HIGHWAY SECTION NO.	02-05 (200-06)
	LENGTH AND HIGHWAY SECTION BRIDGE LOCATION km	
	PROVINCE	Bursa
	HIGHWAY	İnegöl - Bozüyük
PASSED	WATER NAME AND THE RIVER OR SEA IT FALLS IN	Candır
	HIGHWAY NAME	
	RAILWAY NAME	
BRIDGE	KIND	R/C
	TYPE	Simple Girder
	SLAB LENGTH (m)	113.85
	LARGEST SPAN BETWEEN SUPPORTS (m)	15.70
	DATE OF FIELD DATA COLLECTION : 9/5/73	FILM No: 598 POSE No: 5-7
Data Collectors	NAME - SURNAME - POSITION	Ekrem Demirciöz

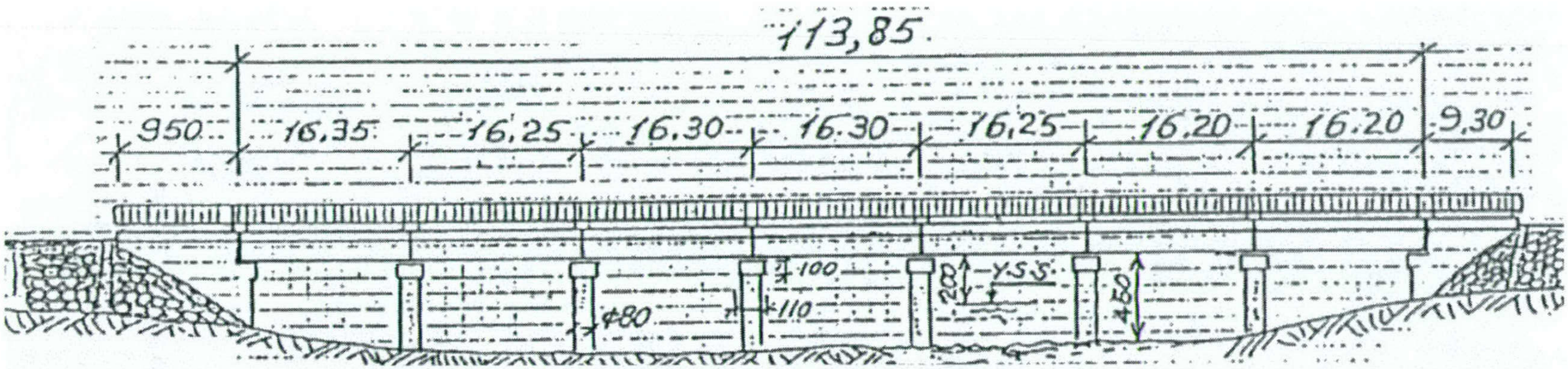
GENERAL INFORMATION ABOUT BRIDGE	SKEWNESS	15°
	KIND OF SLAB	R/C
	KIND OF GIRDER	R/C
	PIER STRUCTURE	R/C Column
	FOUNDATION STRUCTURE	
	APPROXIMATE TONAGE	H20-S16 | Concr.: Wb25 = 225 t/m² | Steel: St 37
	NEW ☒ GOOD ☒	OLD ○ MEDIUM ○ HISTORICAL ○ BAD ○
	IS REPAIR NEEDED?	YES ☐ NO ☒
	IF REPAIR IS NEEDED, ITS LOCATION.	

SCHEMATIC VIEW OF THE BRIDGE



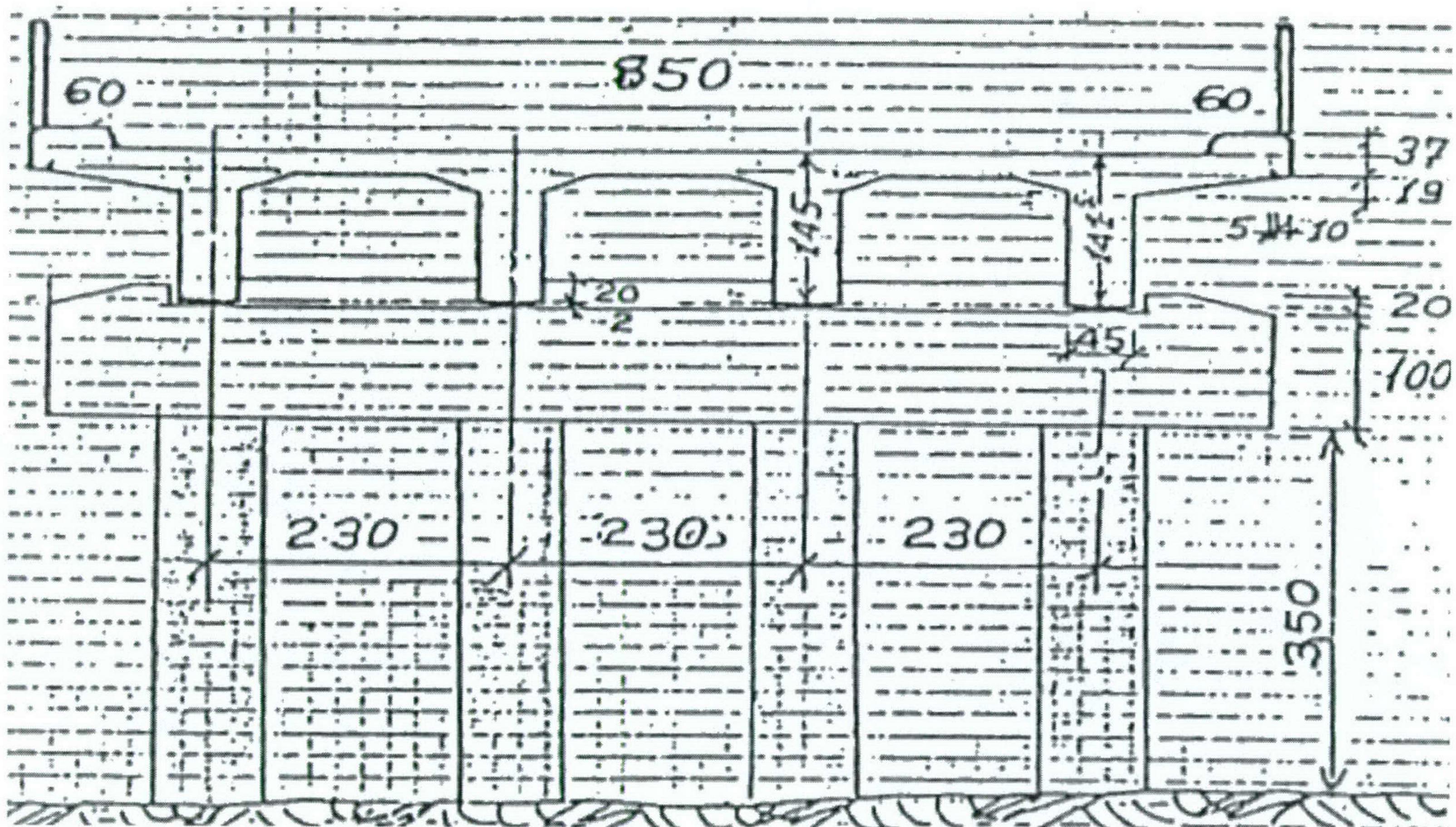
NEW BRIDGE NECESSARY?	YES ○ NO ☒

Figure A1. Bridge Inventory Card, Card 1-Page 2 (Japan International Cooperation Agency 1996)



General Notes:											
High Water Level	:Figure	Low Water Level	Figure	Water Area Sufficient				: Yes	☒	No	☐
Scour in the Foundation	:Yes	☐	No	☒			If yes, is repair urgent?	: Yes	☐	No	☐
Fortification Condition	:Good	☒	Medium	☐	Bad	☐	Is repair urgent?	: Yes	☐	No	☐
Riprap Condition	:Good	☒	Medium	☐	Bad	☐	Is repair urgent?	: Yes	☐	No	☐
Riprap required?	:Yes	☐	No	☒			Is repair urgent?	: Yes	☐	No	☐
Support Conditions	:Good	☒	Medium	☐	Bad	☐	Is repair urgent?	: Yes	☐	No	☐
Rocker Conditions	:Good	☐	Medium	☐	Bad	☐	Is repair urgent?	: Yes	☐	No	☐
Girder Condition	:Good	☒	Medium	☐	Bad	☐	Is repair urgent?	: Yes	☐	No	☐
Slab Condition	:Good	☒	Medium	☐	Bad	☐	Is repair urgent?	: Yes	☐	No	☐
Rebar Exposed?	:Yes	☐	No	☒		☐	Gullies work?	: Yes	☒	No	☐
Balustrades and Steel Parts	:Good	☒	Medium	☐	Bad	☐	Paint condition	: Painted	☒	Not Painted	☐
Other Notes:											

Figure A2. Bridge Inventory Card, Card 1-Page 2 (Japan International Cooperation Agency 1996)



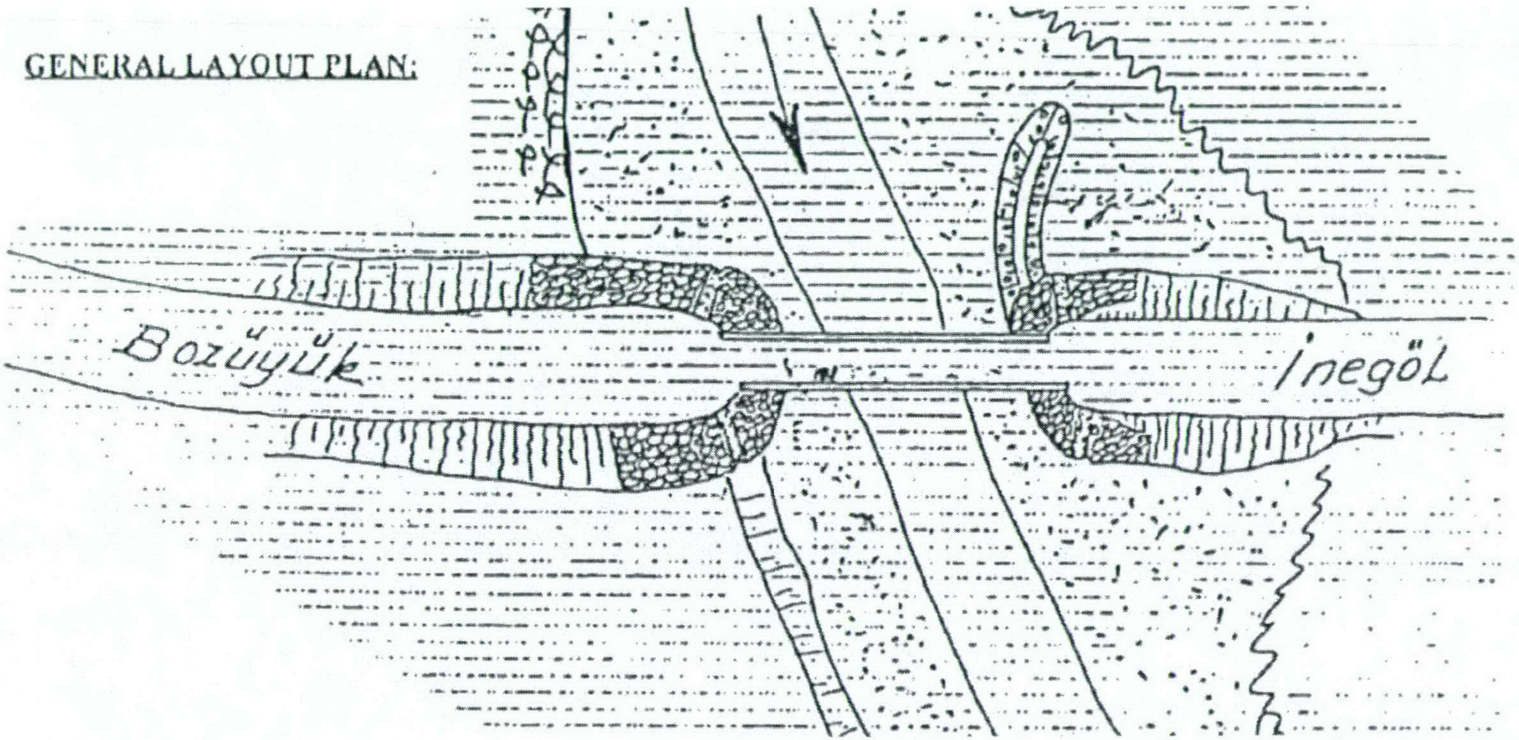
Slab width: 60 + 850 + 60 Horizontal Gauge : 9.45 Vertical Gauge : —

Slab cover : Skewness : 15 Superelevation : —

DATA COLLECTORS: Position: Name Surname: Ekrem Demirci

Figure A3. Bridge Inventory Card, Card 2-Page 1 (Japan International Cooperation Agency 1996)

GENERAL LAYOUT PLAN:



General Notes:

Information about river bed:

Side passing	Easy	☒	Hard	☐	Impossible	☐						
Stone pitching condition	Good	☒	Medium	☐	Bad	☐	, Return wall condition:	Good	☒	Medium	☐	Bad ☐
Distructives?	Yes	☐	No	☒								
Training wall condition	Good	☐	Medium	☐	Bad	☐	, Dike conditions:	Good	☐	Medium	☐	Bad ☐

(Flow direction, Fortification, Embankments and Slopes, Skewness, Presence on Alignment or Curve, Access Road Curve Radius, City Names in Both Directions of the Bridge Road will be written on General Layout Plan).

Figure A4. Bridge Inventory Card, Card 2-Page 2 (Japan International Cooperation Agency 1996)

Appendix B
Trucks

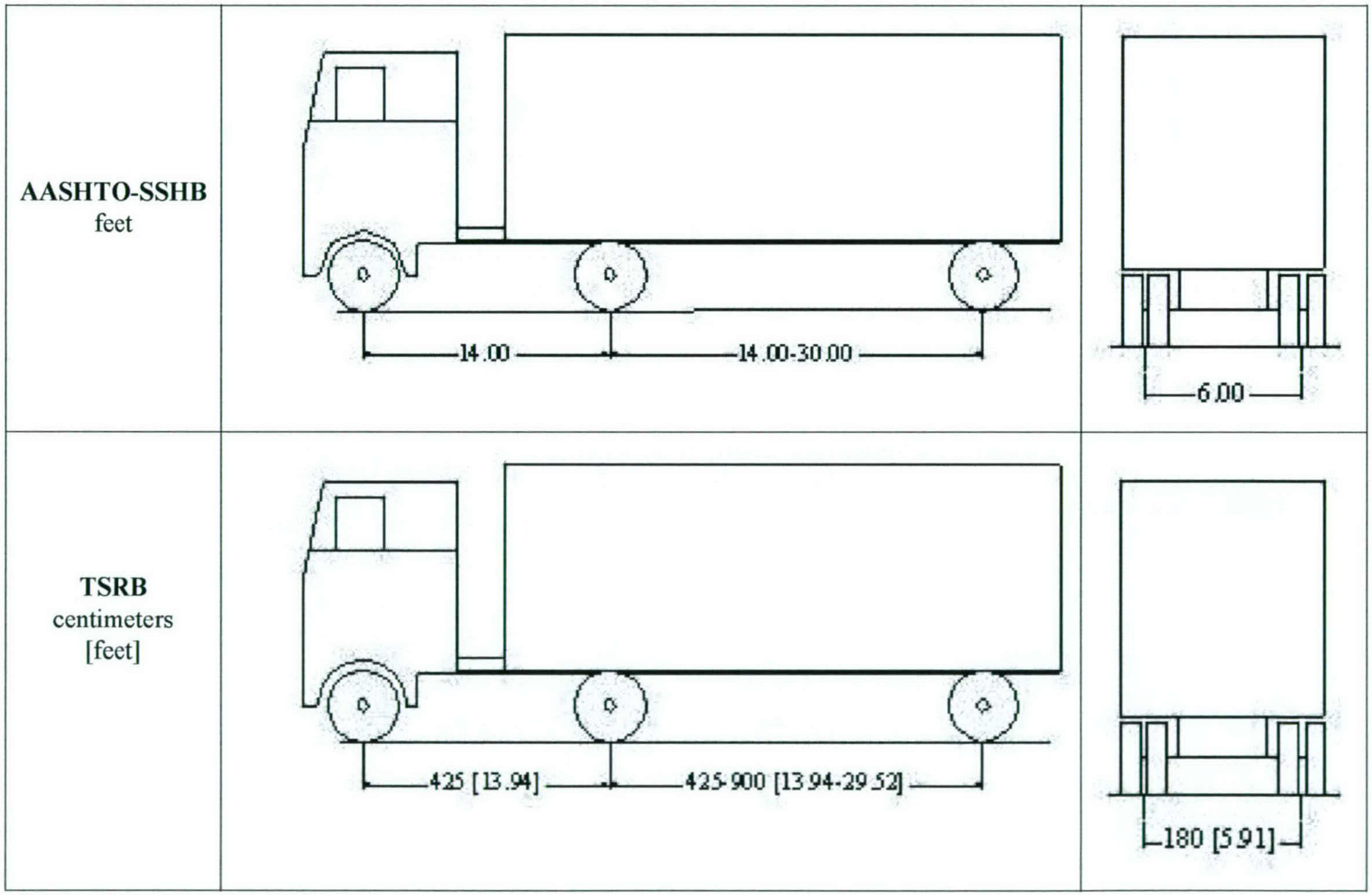


Figure B1. HS Truck Geometry (AASHTO 1996; GDH 1982)

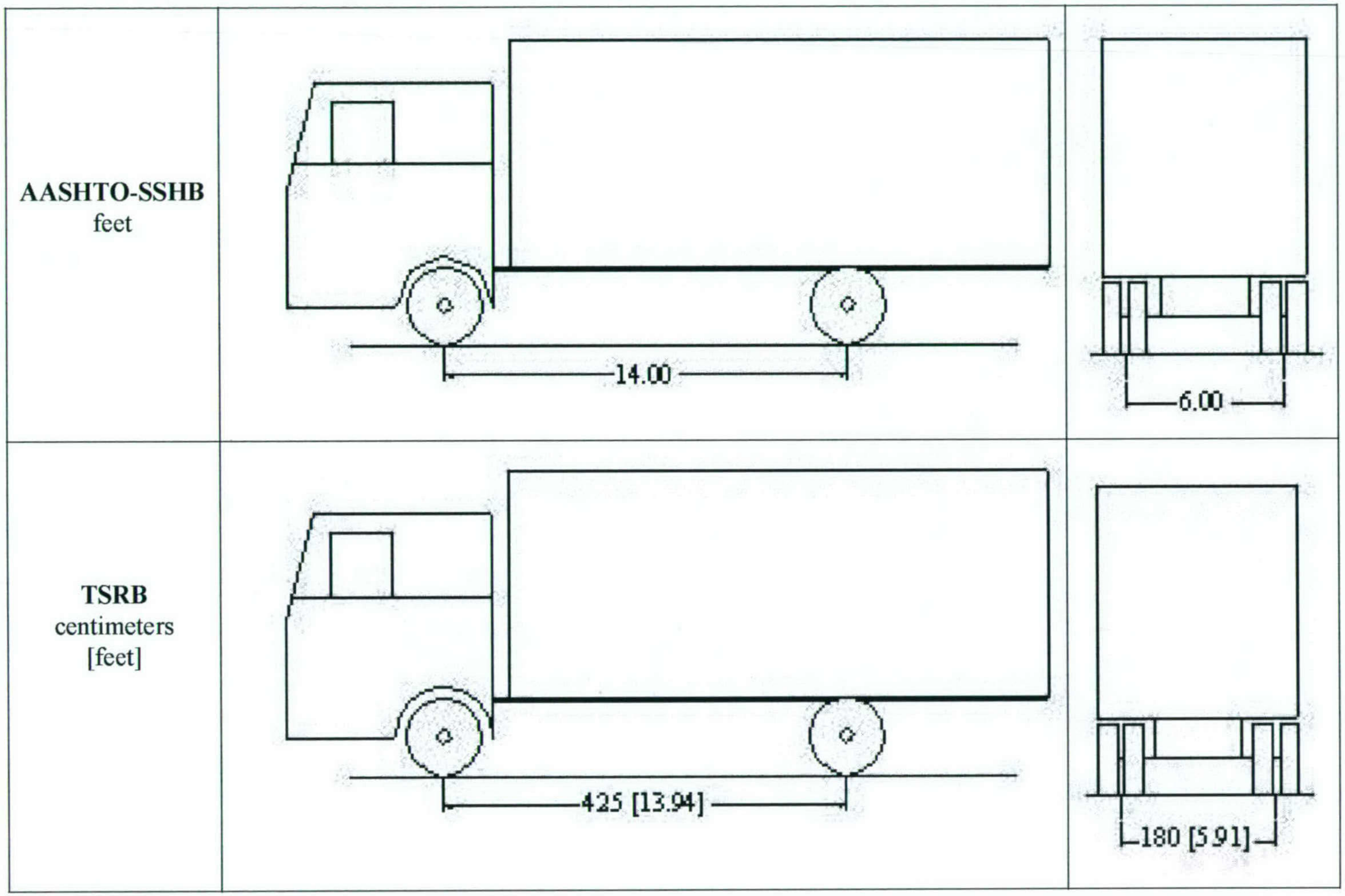


Figure B2. H Truck Geometry (AASHTO 1996; GDH 1982)

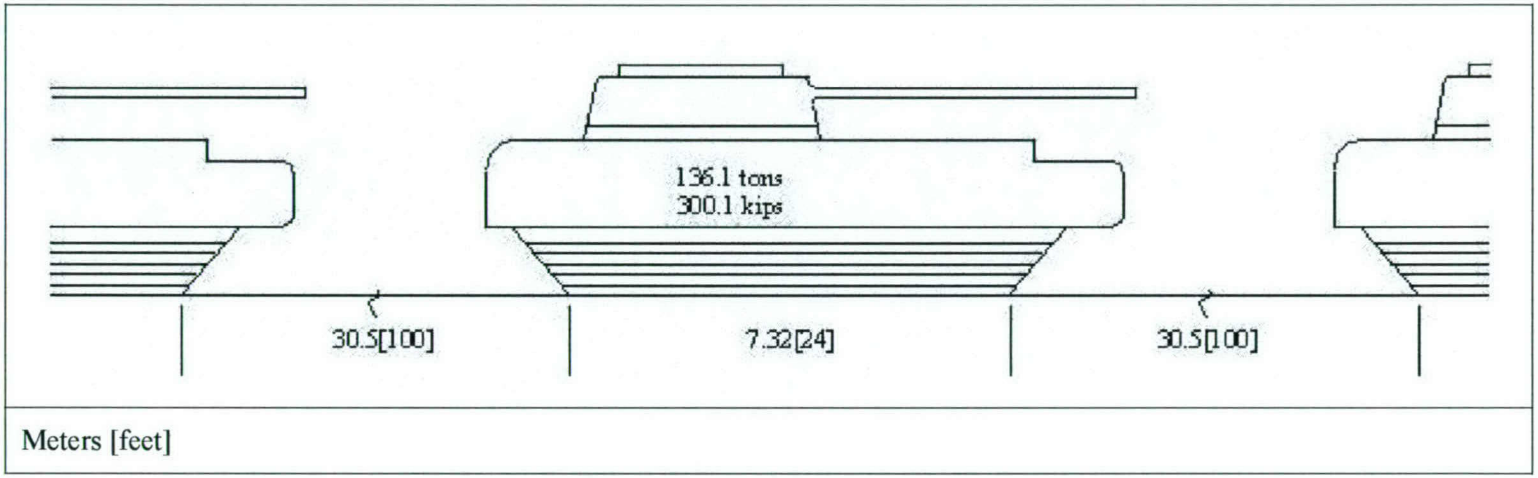


Figure B3. Overloading Type-A for Turkish expressways (GHD 1997)

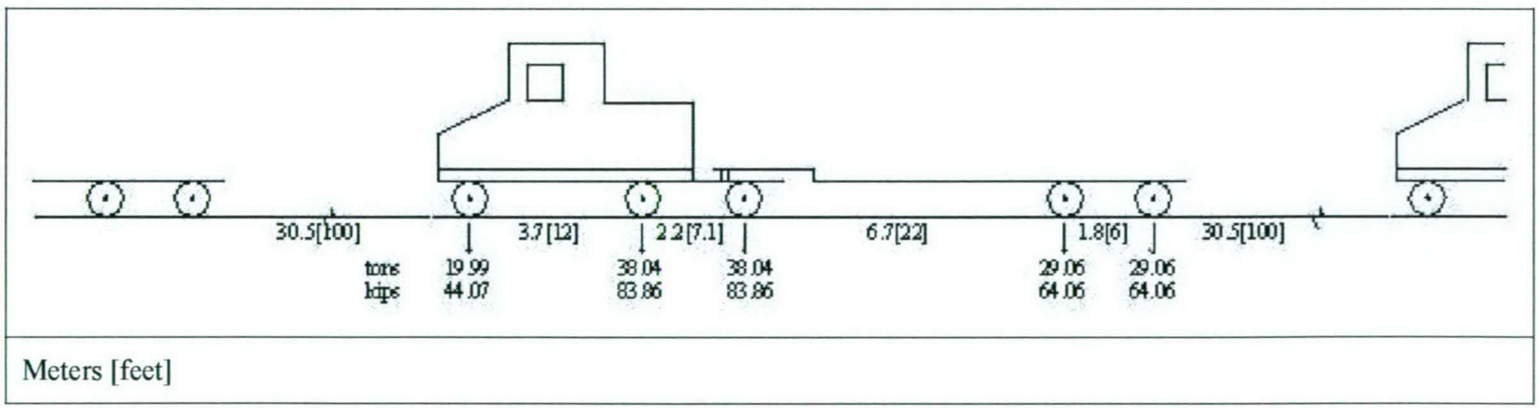


Figure B4. Overloading Type-B for Turkish expressways (GDH 1997)

Appendix C
Analysis Diagrams of Birecik Bridge

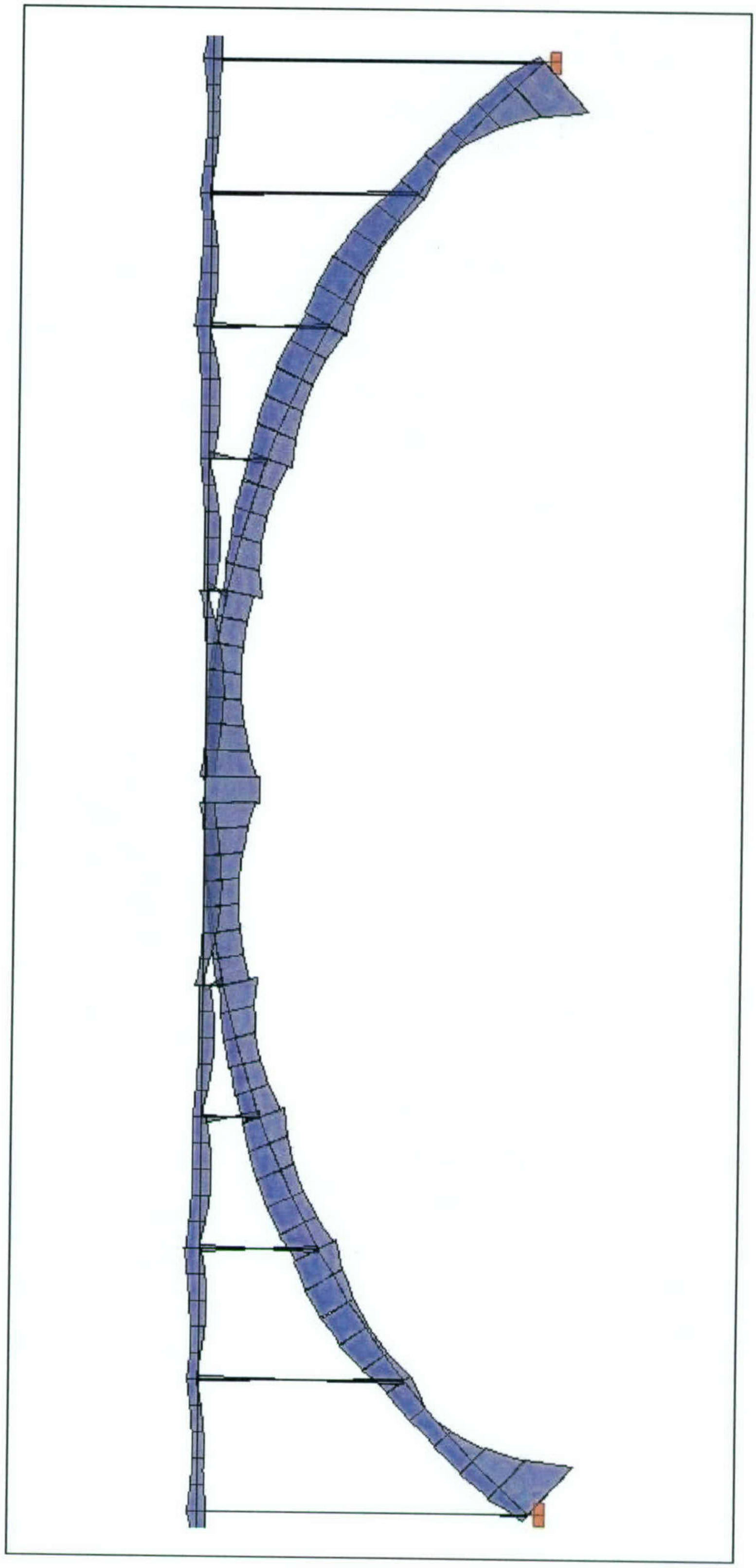


Figure C1. Moment diagram of the main arch, Birecik Bridge

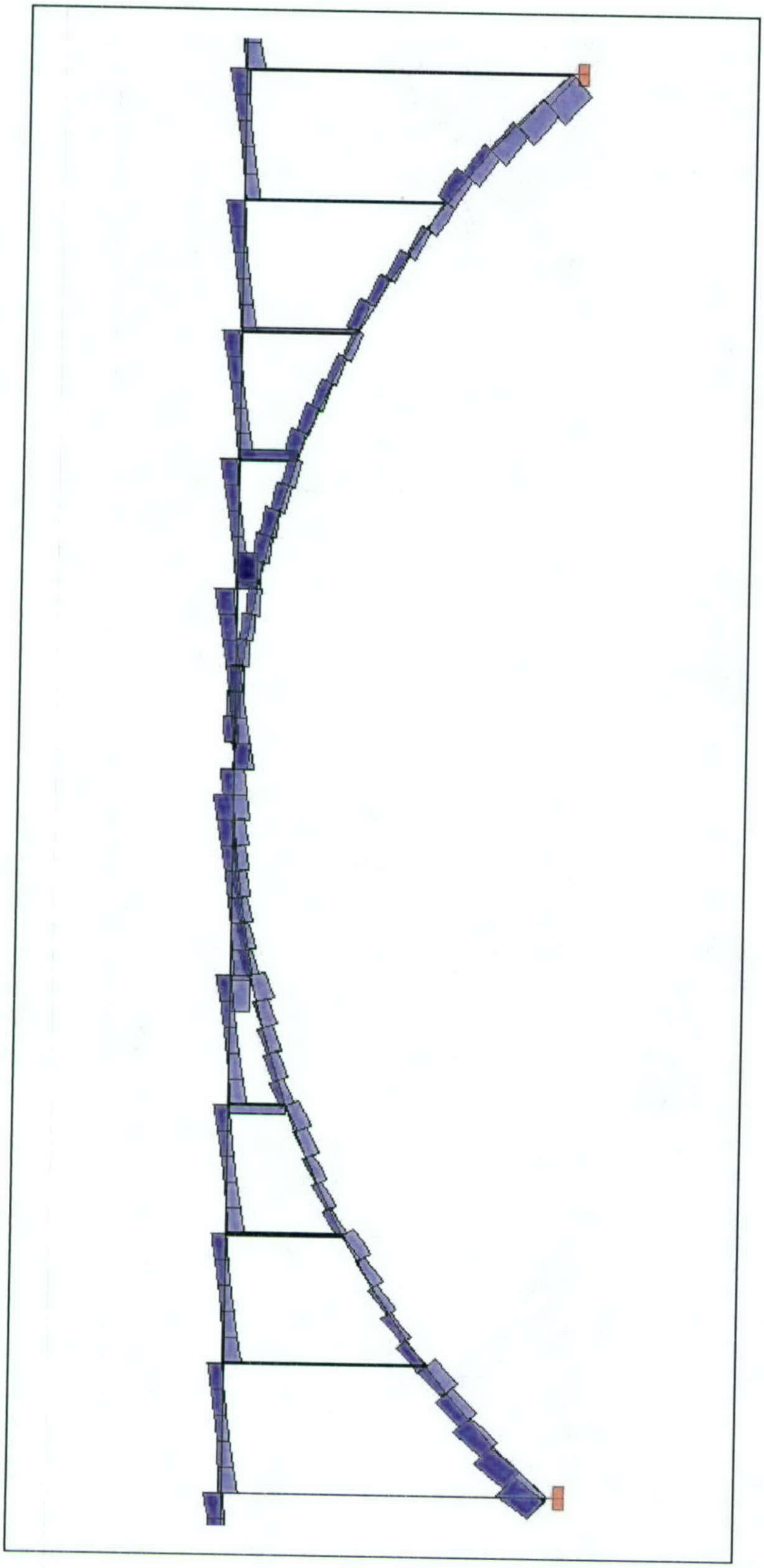


Figure C2. Shear force diagram of the main arch, Birecik Bridge

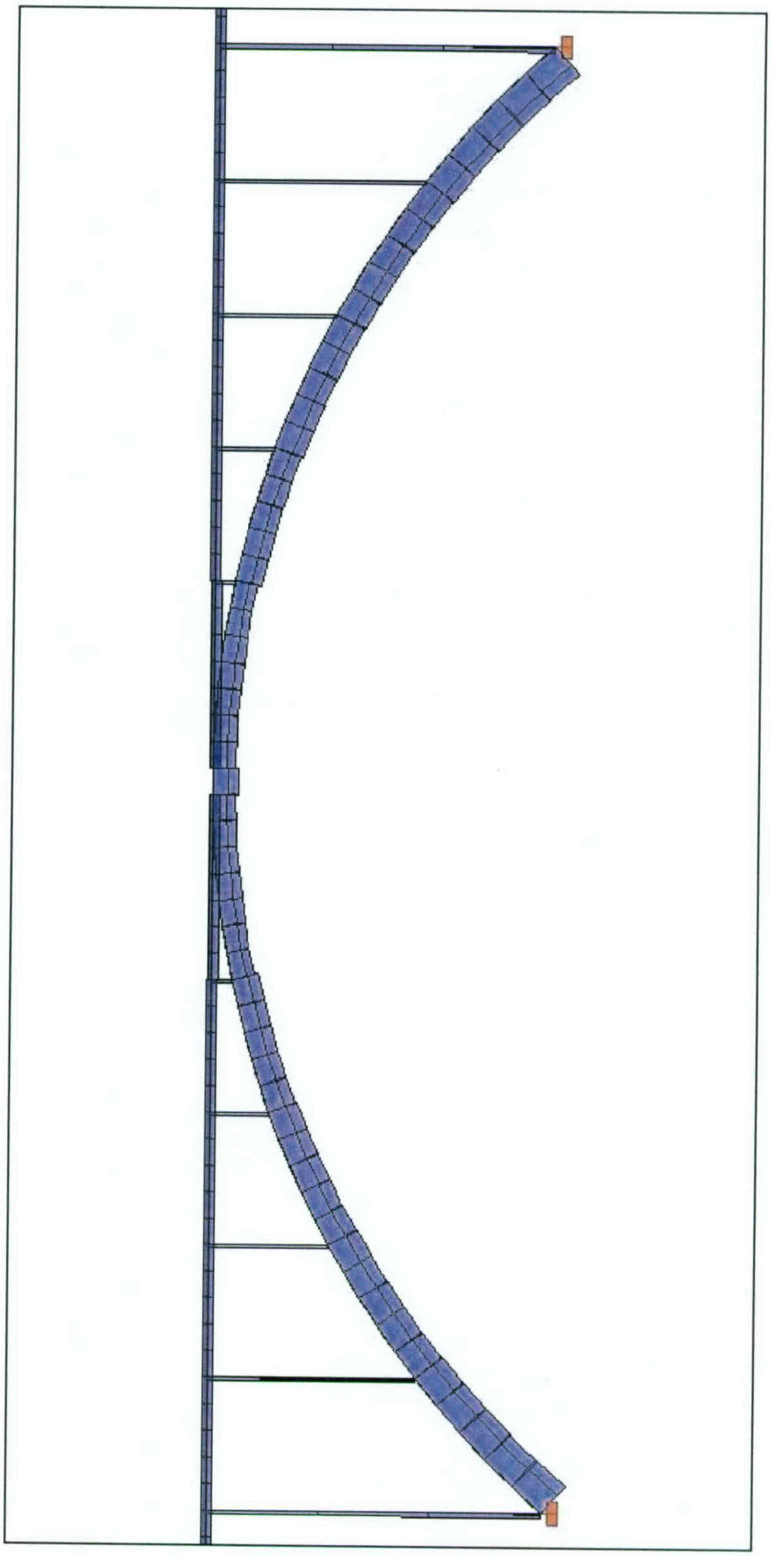


Figure C3. Axial force diagram of the main arch, Birecik Bridge

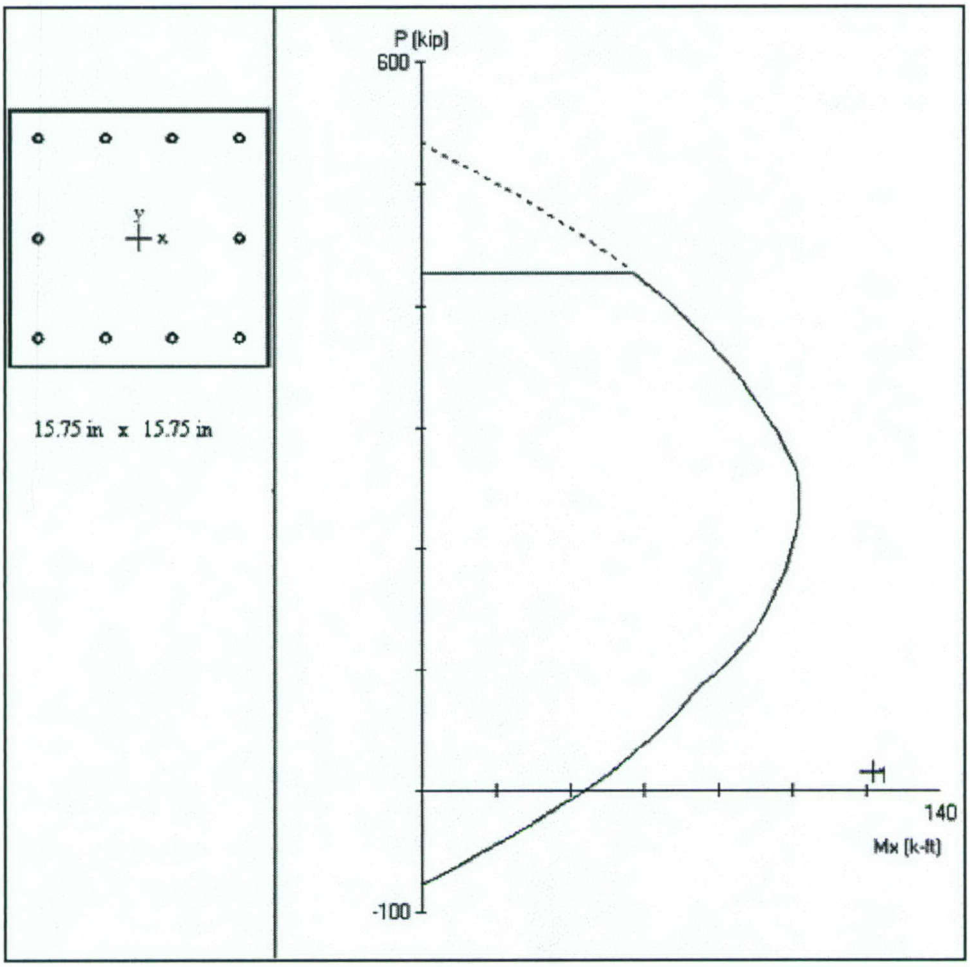


Figure C4. Interaction diagram of the Column 4 for the minimum steel (1 percent)

Plus sign shows the point of the load pair on the diagram. The plus sign should be inside of the envelope to call it as a safe column.

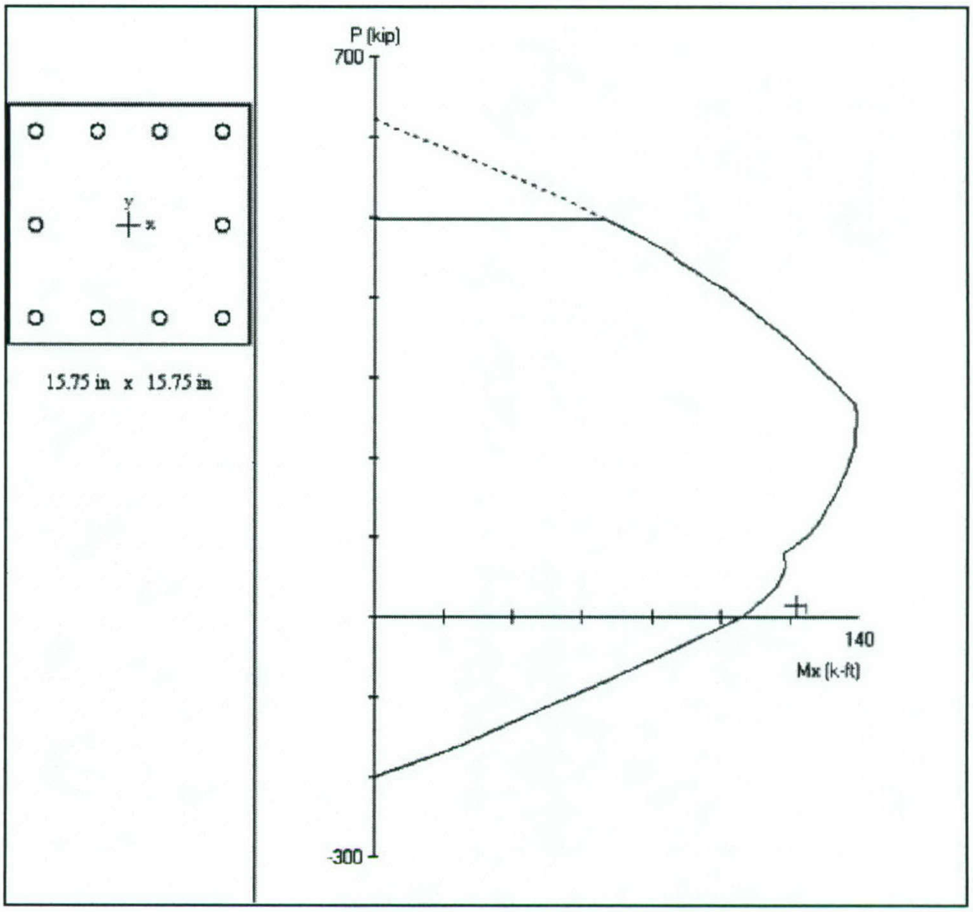


Figure C5. Interaction diagram of the Column 4 with 2.64 percent steel

This second assumption of steel ratio based on the information obtained from the blueprints of Dokuzdolambac II Bridge. In this diagram the steel ratio was 2.64 percent. The column was still unsafe as the plus sign was out of the envelope.

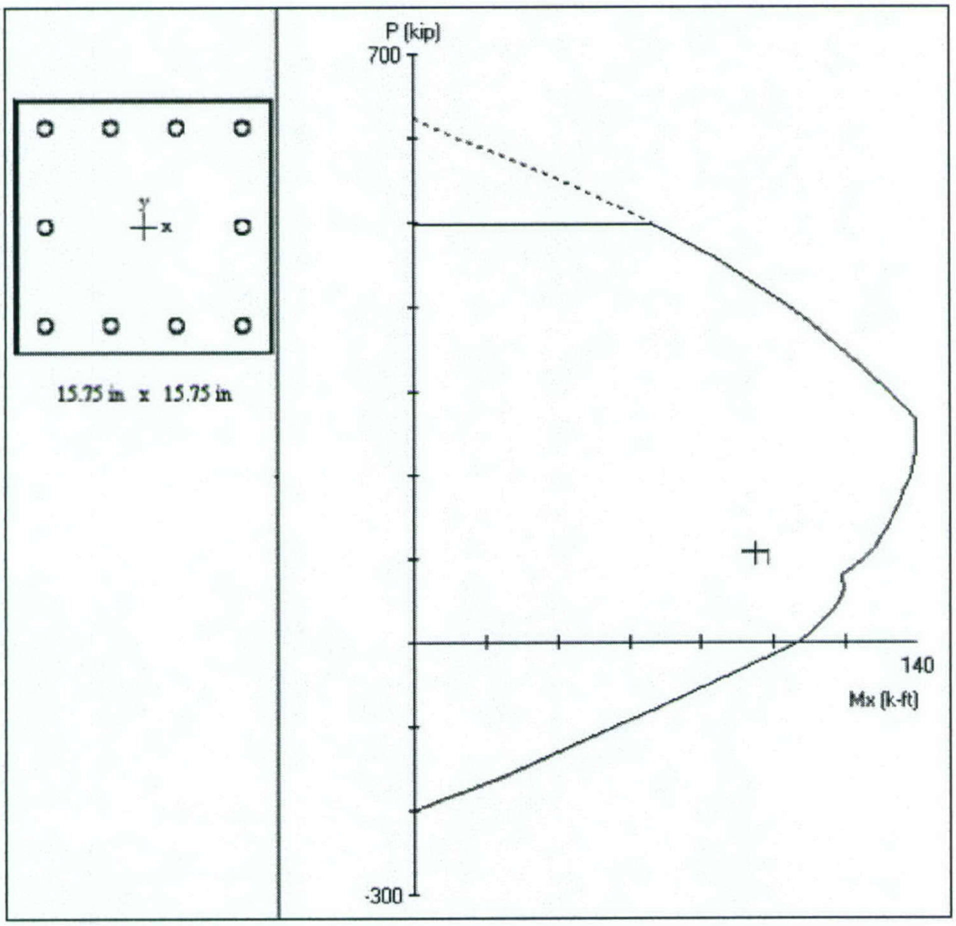


Figure C6. Interaction diagram of the Column 2 (slender) with 2.64 percent steel

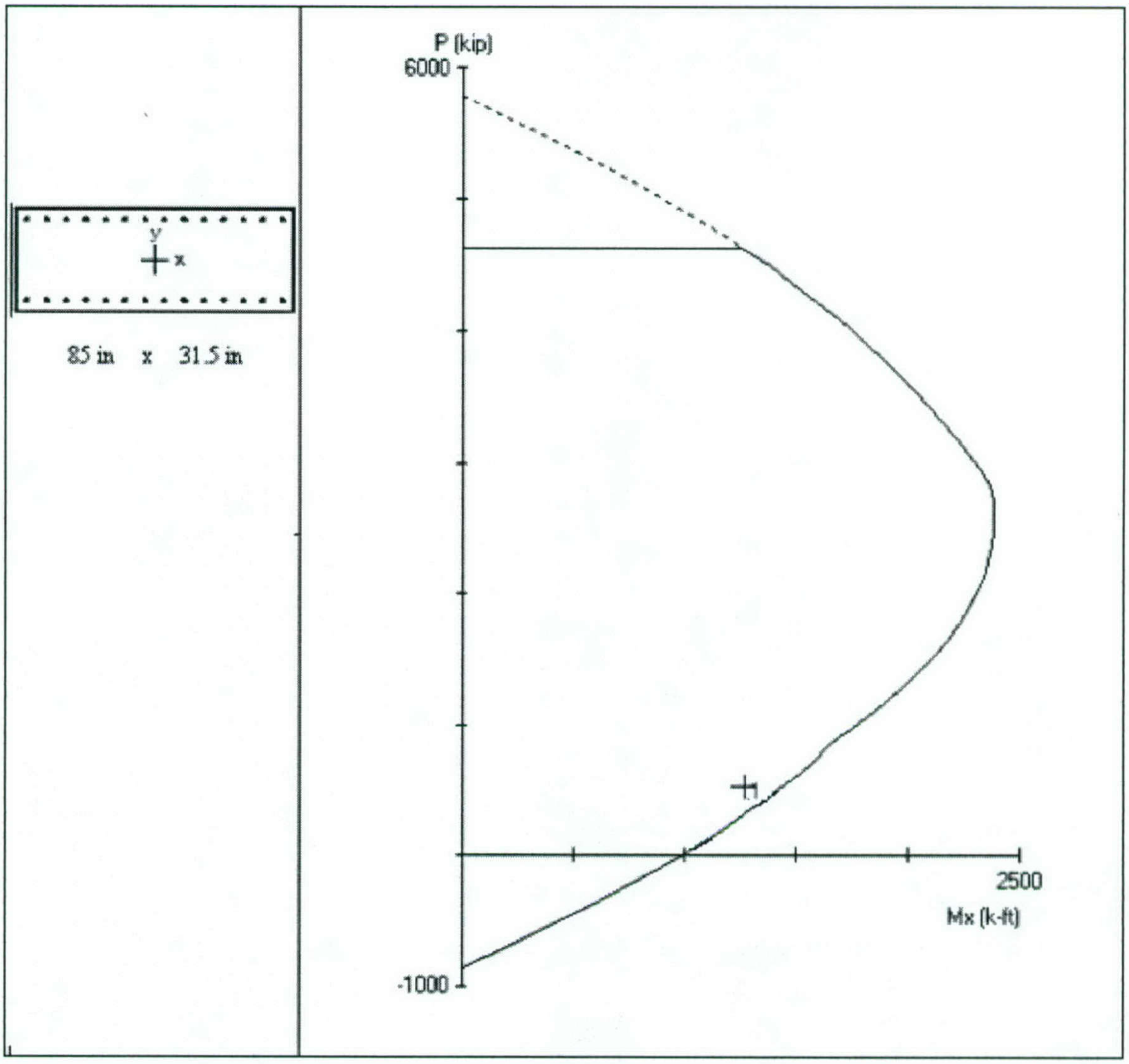


Figure C7. Interaction diagram of the arch for the minimum steel

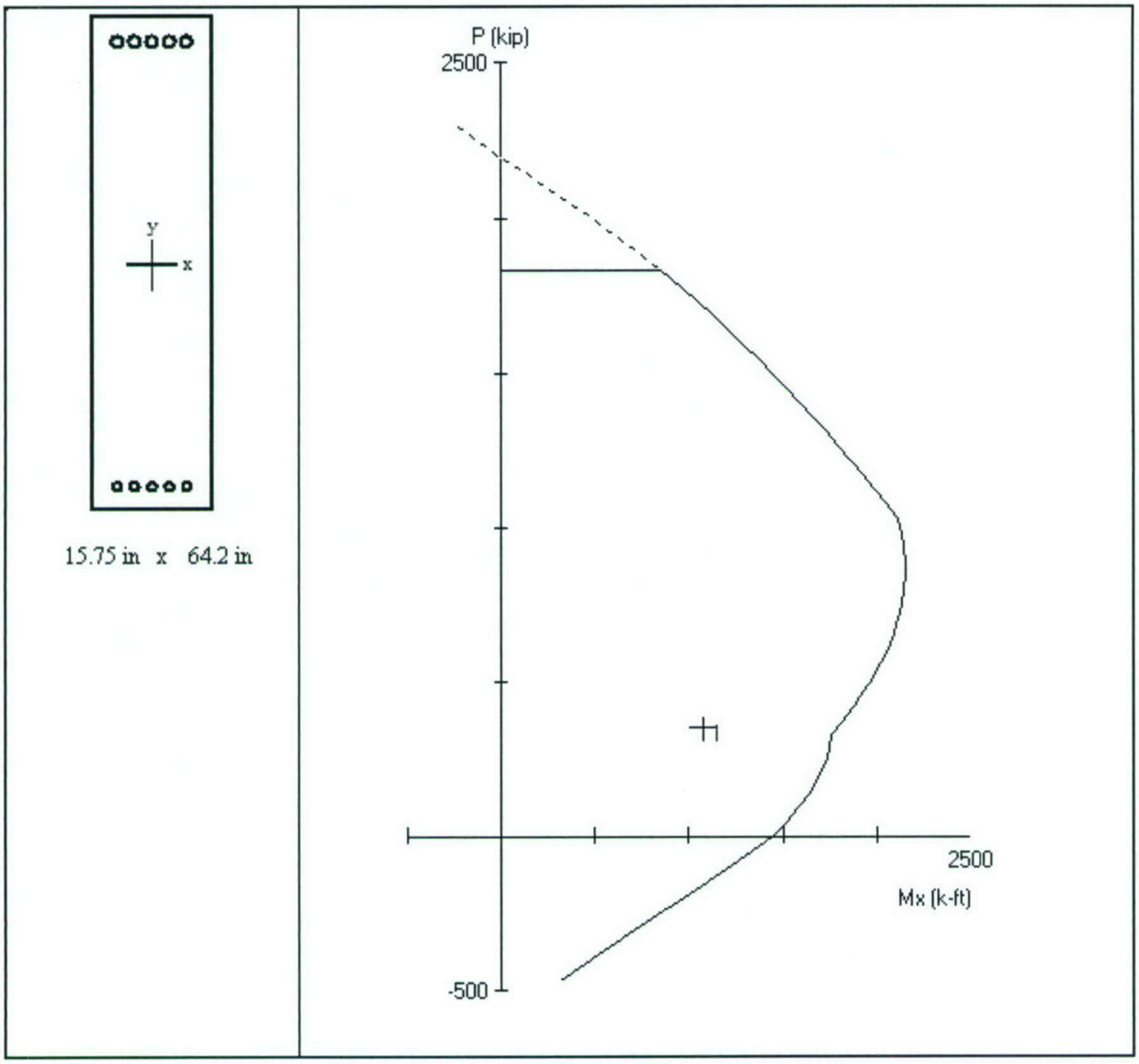


Figure C8. Interaction diagram of the arch-crown for the minimum steel

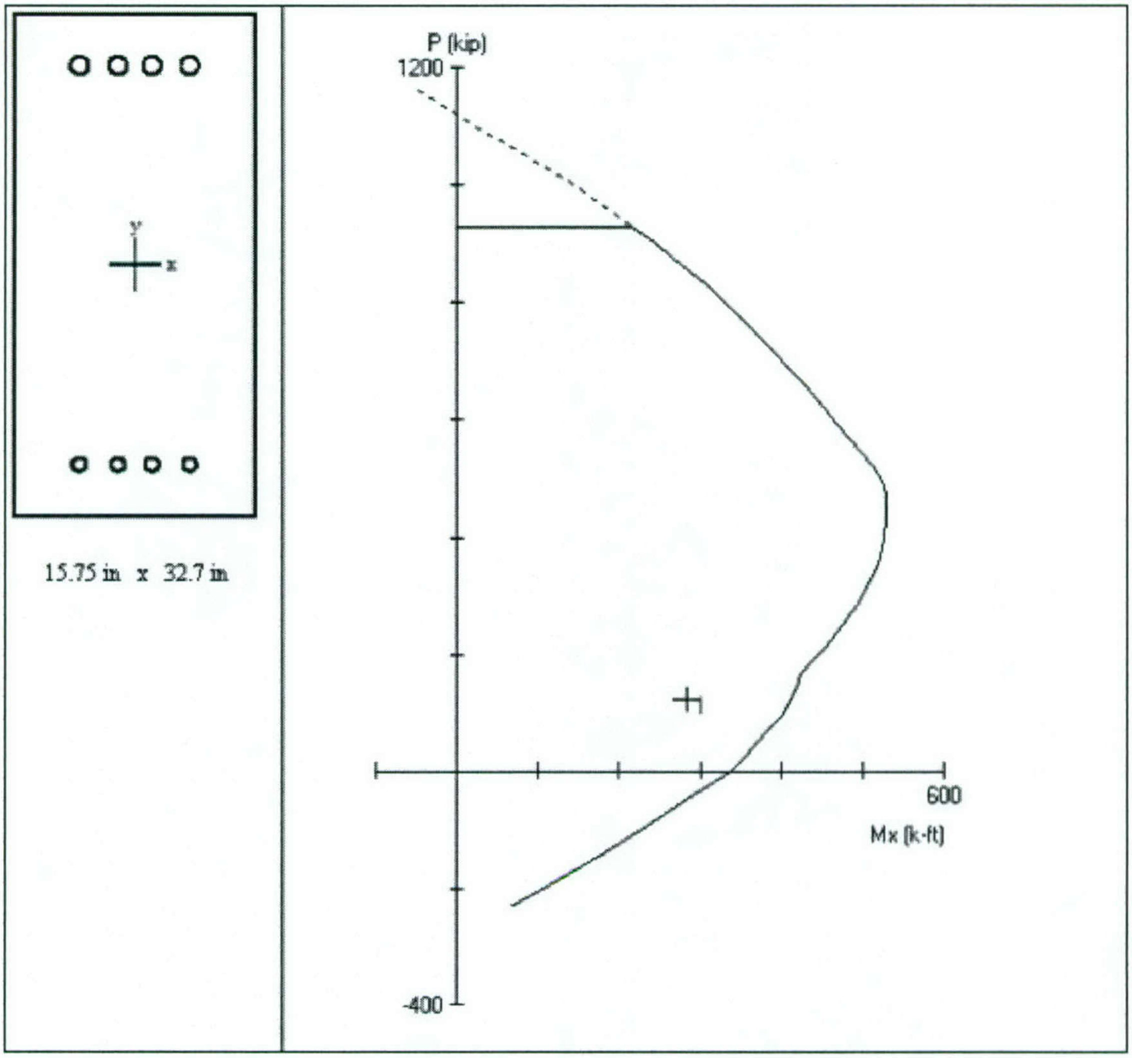


Figure C9. Interaction diagram of the girder for the minimum steel

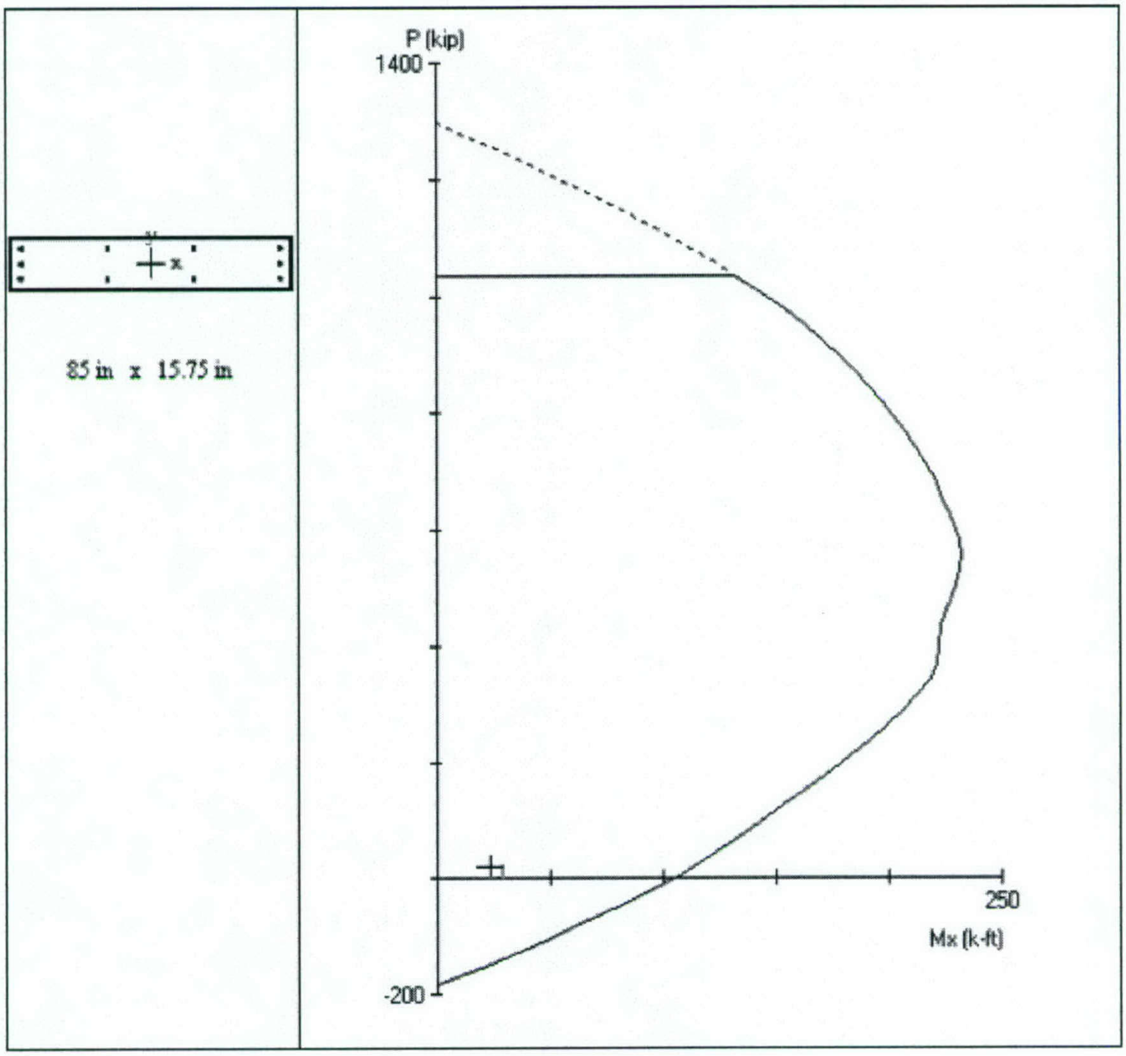


Figure C10. Interaction diagram of the column-wall for the minimum steel

Appendix D
Pictures of Birecik Bridge
February 2003

Figure D1. The traffic on the Birecik Bridge

Figure D2. Side view of the Birecik Bridge

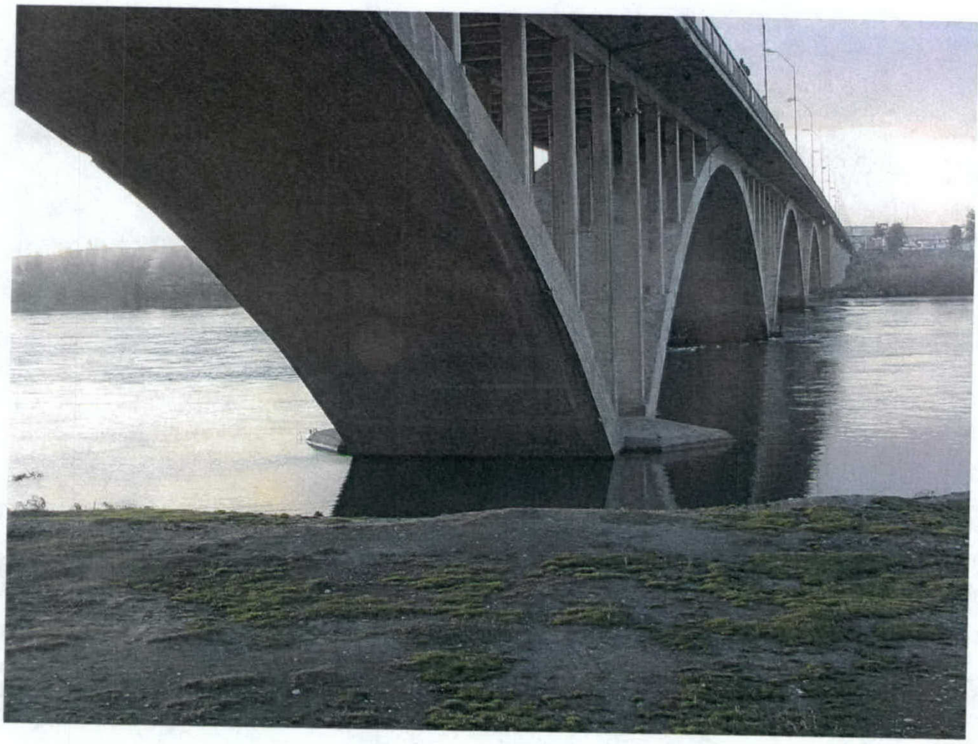


Figure D3. Side view of the Birecik Bridge and the Euphrates River

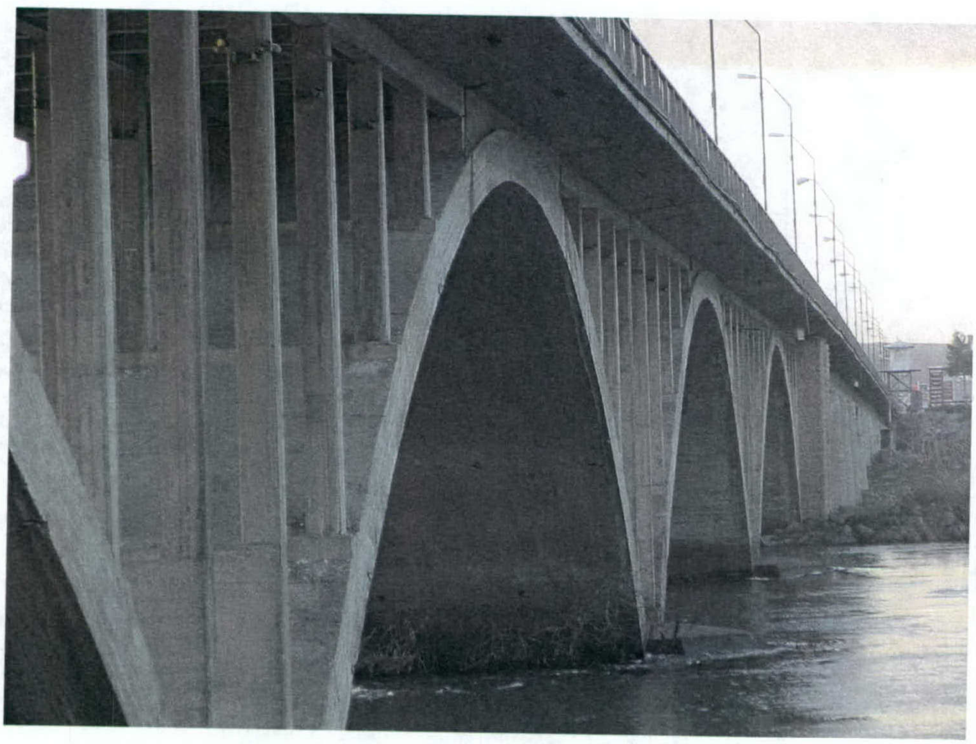


Figure D4. Side view of the arches of the Birecik Bridge

Figure D5. The arches of the Birecik Bridge on the land

Figure D6. The arch rib of the Birecik Bridge

Figure D7. The deck of the Birecik Bridge

Figure D8. A detail of column-beam-diaphragm connection of the Birecik Bridge

Figure D9. The arches, and the column wall of the Birecik Bridge

Figure D10. The exterior girder and the diaphragm of the Birecik Bridge

Appendix E
Pictures of Candir Bridge
May 2003

Figure E1. The approach of the Candir Bridge

Figure E2. Side view of the Candir Bridge

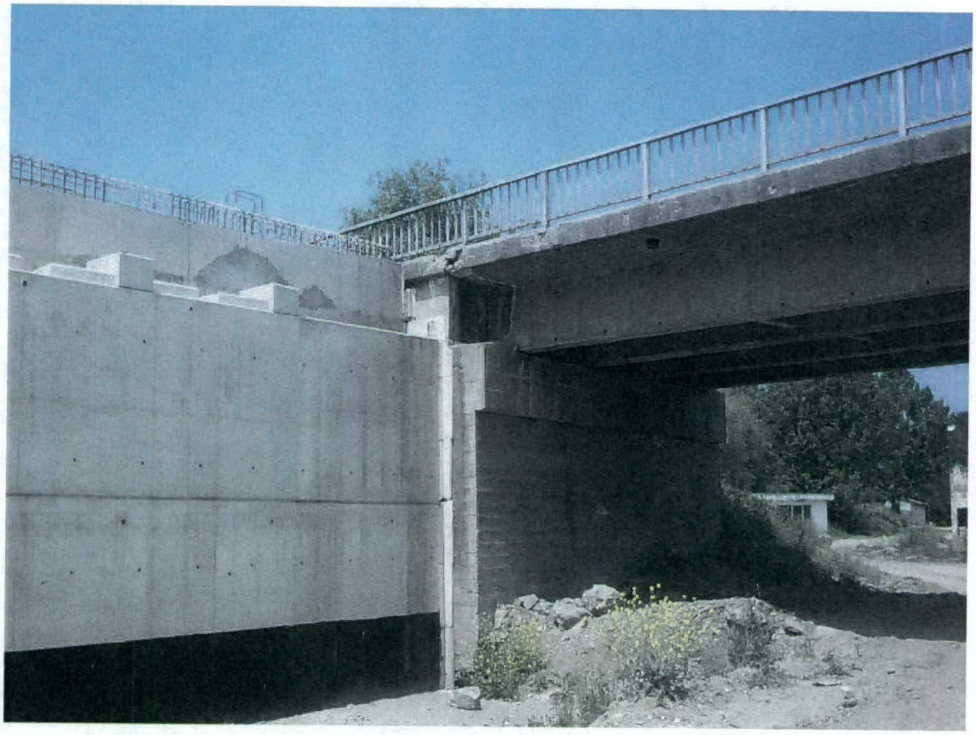


Figure E3. Abutments of the Candir Bridge and the new bridge

Figure E4. The Candir River in the summer

Figure E5. The deck and the pier cap of the Candir Bridge

Figure E6. A detail of the deck of the Candir Bridge

Figure E7. The exterior girder of the Candir Bridge

Figure E8. The simple span support of the Candir Bridge

Figure E9. The first support of the Candir Bridge from the west

Figure E10. The girder of the Candir Bridge resting on an abutment

Figure E11. Construction of the new bridge

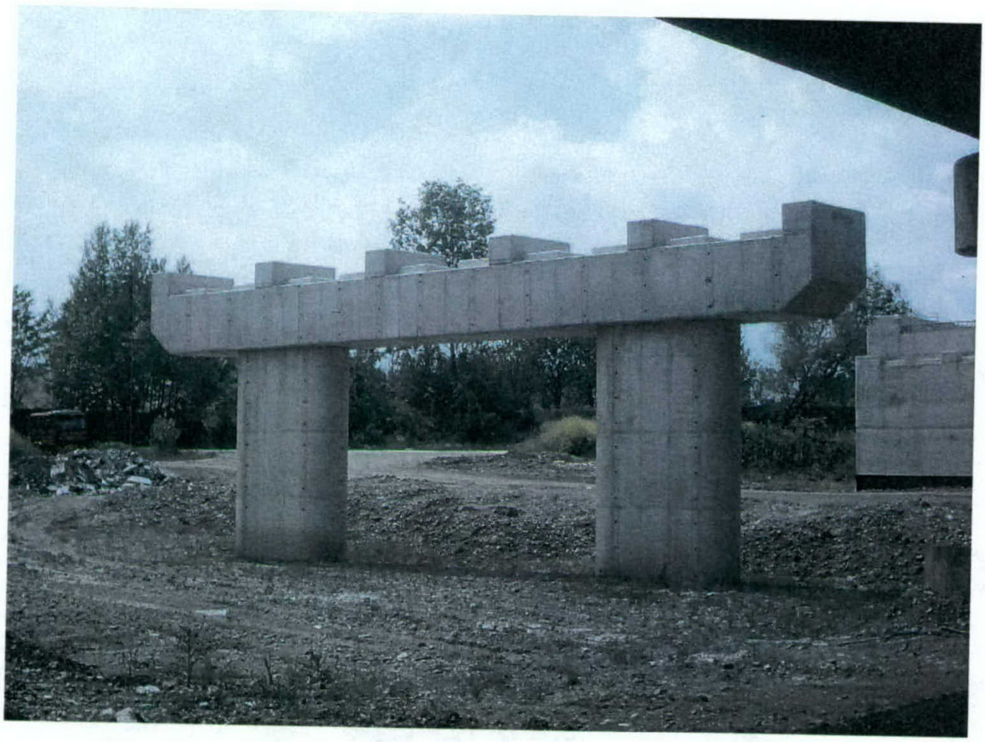


Figure E12. The pier of the new bridge

Appendix F
Bridge Information Sheets

Candir Bridge

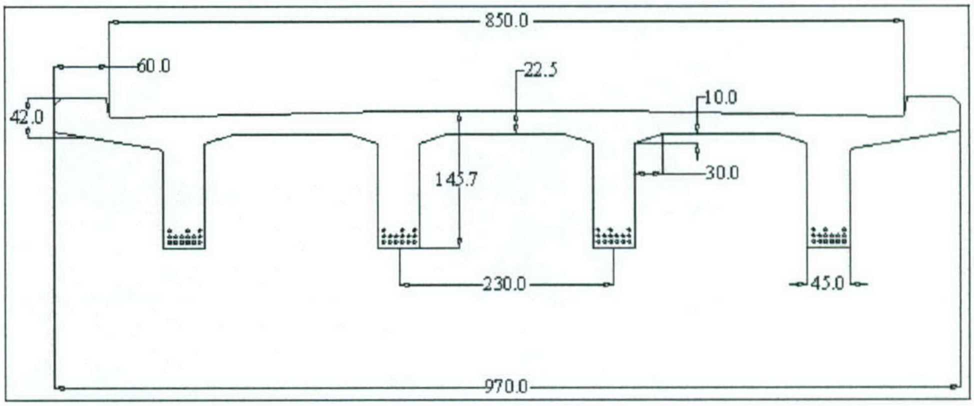


Figure F1. Cross-section details of Candir Bridge, cm

Reinforced concrete T-Girder Bridge
Two-lane Bridge on a state highway
Northwestern Turkey
Constructed in 1972
7 simple spans
Max. Span Length = 15.7 m
Total Length = 113.5 m
Design Live Load = HS20 (HS22 of AASHTO)

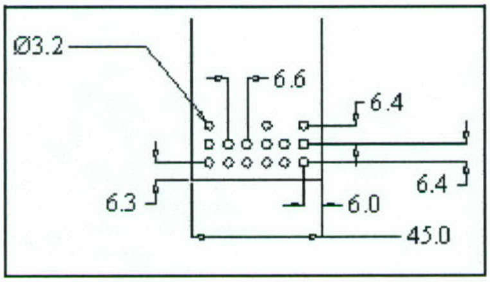


Figure F2. Reinforcement details, cm

Deck Width	970 cm
Curb-to-Curb Width	850 cm
Deck Thickness	22.5 cm
Number of T-Beams	4
Constant T-Beam Depth	145.7 cm
T-Beam Thickness	45 cm
Beam Spacing	230 cm
Curb Height	42 cm
Left Curb Width	60 cm
Right Curb Width	60 cm
Concrete Strength	22.1 MPa
Reinforcement per Beam	120.6 cm^2
Depth of Reinforcement	134.6 cm
Yield Stress	234.4 MPa

I-Girder Bridge

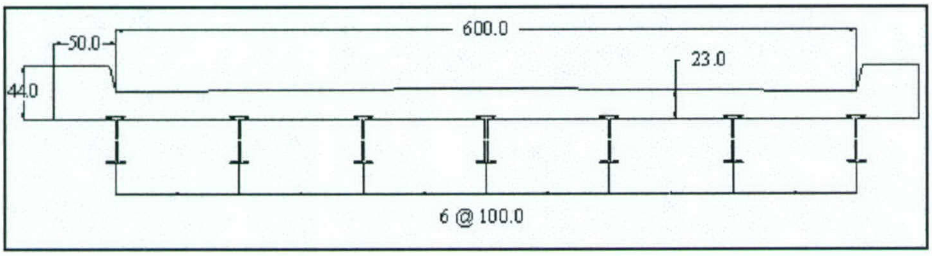


Figure F3. Cross-section details of I-Girder Bridge, cm

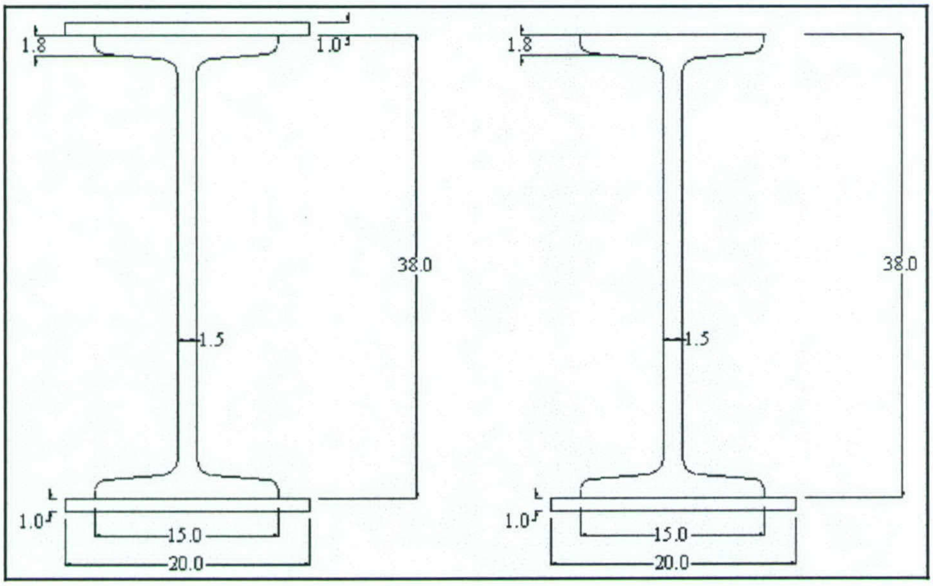


Figure F4. Girder at support, cm

Figure F5. Girder at midspan, cm

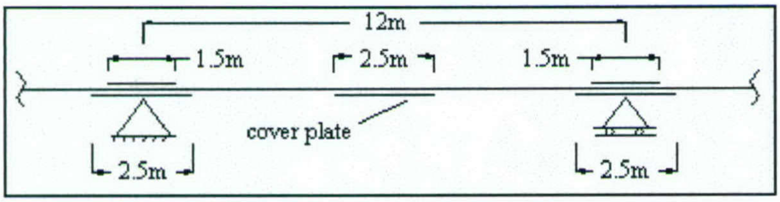


Figure F6. Placing of cover plates

Generic bridge drawings prepared before 1950s
Reinforced concrete slab- steel I-Girder continuous span

Max span	12.00 m
Concrete Strength	22.1 MPa
Yield Stress	234.4 MPa
Design Live Load	HS15 (HS16.5 of AASHTO)

I-Girder is a standard section with a depth of 38 cm

Birecik Bridge

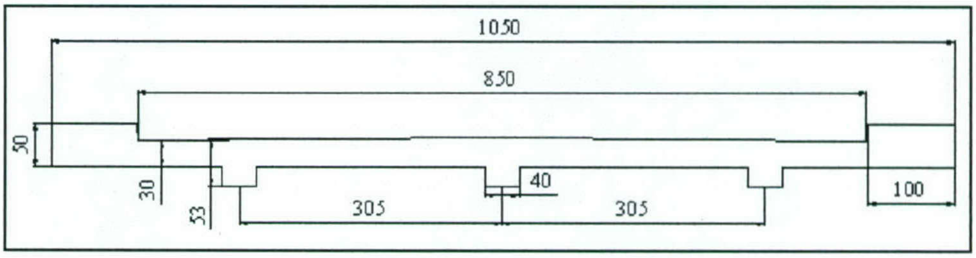


Figure F7. Cross-section details of Birecik Bridge, cm

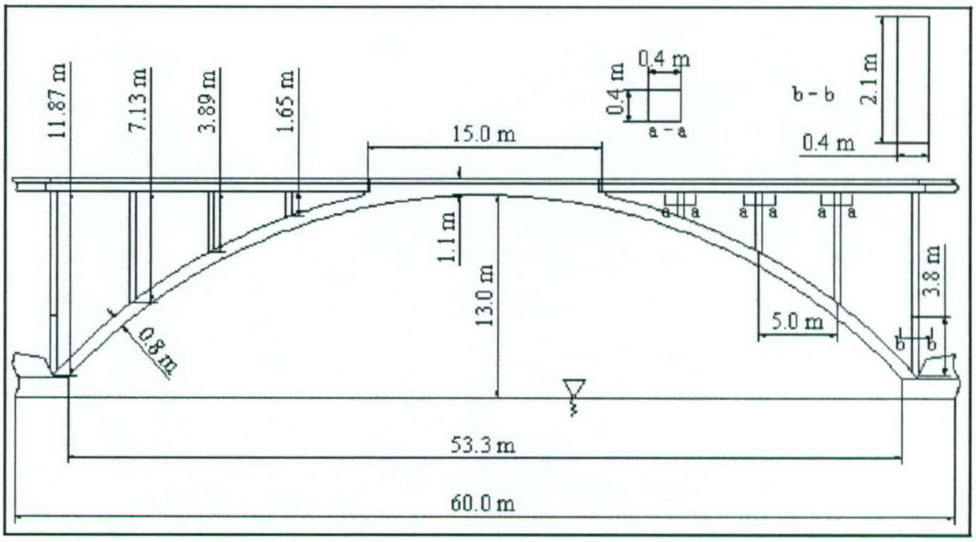


Figure F8. Dimensions of the arch of Birecik Bridge

Reinforced concrete open spandrel arch bridge with 5 identical arches
Constructed in 1956
Two-lane bridge on a state highway
Southeastern Turkey
Reinforcement information not available

Concrete Strength	= 22.4 MPa
Yield Stress	= 234.4 MPa
Design Live Load	= HS20 (HS22 of AASHTO)

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13. SUPPLEMENTARY NOTES

14. ABSTRACT

Turkish bridge design standards were studied, with attention focused on the live load. The design specifications were compared with American design specifications. The major difference was that the live load in Turkish standards is given in tonnes; whereas, in American Association of State Highway and Transportation Officials-Standard Specifications for Highway Bridges (AASHTO 1996) it is in tons. Therefore, HS20 in Turkish standards is 10 percent heavier than HS20-44. Turkish bridges are currently designed to either HS20 or HS30, the latter being 65 percent heavier than HS20-44. There were some minor differences in other requirements, due to conversion from United States customary units to metric units.

Three types of Turkish bridges were analyzed using a service load approach according to AASHTO (1996) using a Heavy Equipment Transporter as the live load. Service load approach was applied. Only the primary loads, dead load, live load, and impact were considered. The analysis did not include any modification for possible deterioration, damage, or aging of the bridges.

15. SUBJECT TERMS

Bridge dead load	Bridge live loads	Turkish bridge standards
Bridge design	Bridge rating	Turkish concrete properties
	Turkey bridge design	

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